IRC:SP:143-2024

# GUIDELINES ON STEEL CONCRETE COMPOSITE BOX GIRDER SUPERSTRUCTURE FOR ROAD BRIDGES

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## Guidelines on Steel Concrete Composite Box Girder Superstructure for Road Bridges

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# 1. INTRODUCTION

1.1. Steel-concrete composite box girders offer advantageous structural solution over other superstructure types in terms of span range and torsional stiffness. It is often economical in span ranges encountered in flyovers at their intersection with roads below. Due to their large torsional stiffness, the box girder bridges offer economical solutions in curved bridges. Box girders are suitable for longer span ranges than I-girders and allow larger span to depth ratio which is advantageous in urban flyovers in congested areas. Being light weight, handling and erection are easier compared to precast girders.

The design, detailing and fabrication of steel-concrete composite box girder superstructures are more complex than steel plate girder superstructures, especially under construction stage condition. IRC:22 "Standard Specifications and Code of Practice for Road Bridges, Section VI – Composite Construction (Limit States Design) (Third Revision)" being the general code for Steel-Concrete Composite Construction does not specifically address all aspects of composite box girder design. Guidelines for Design of Steel-Concrete Composite Box Girder Superstructures for Road Bridges that would address general rules on sizing, spacing and framing plans, various analysis tools, specifics on design of different components of the superstructure.

1.2. Accordingly, the Steel and Composite Structure Committee (B-5) undertook the preparation of draft Guidelines on Steel Concrete Composite Box Girder Superstructure for Road Bridges during the year 2015-2017 which carried over for another two tenures i.e. 2018-2020 & 2021-2023. During the year 2015, the then B-5 Committee constituted a Sub-committee for the preparation of the aforesaid guidelines comprising of Dr. Arul Jayachandran as Convenor & Dr. Satish Kumar, Shri Arijit Guha, Dr. V. Kalyanaraman, Shri Alok Bhowmick, Shri V. Karthik, Shri Sumantra Sengupta and Dr. S. K. Garg as members.

The initial of the draft document was discussed in a number of meetings of the Sub Group and was simultaneously discussed in the meetings of B-5 Committee for necessary modifications during three tenures. The draft document after going through various modifications was discussed at length and was finalized in the 6th meeting of B-5 Committee held on 16.07.2022

The Composition of the Steel and Composite Structures (B-5) Committee is given below:

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The finalized draft document was placed before the Bridges Specifications & Standards Committee (BSS) in its meeting held on 25th September, 2022 and was approved for placing before EC & Council for their consideration. After due deliberation on the draft document the Executive Committee in its meeting held on 7th October, 2022 approved the same for placing the same before the Council. The draft guidelines, after due deliberation was approved by the IRC Council in its Meeting held on 8th October, 2022 at Lucknow (U.P.) for printing.

## 2. SCOPE

This document provides the guideline for design of rectangular or trapezoidal, closed or open (tub), simply supported or continuous, straight or curved steel concrete composite box girder superstructures. The superstructure may have single girder or multiple girders.

The basic design procedure shall be as per IRC:22 "Standard Specifications and Code of Practice for Road Bridges, Section VI – Composite Construction (Limit State Design) (Third Revision)", whereas the design of steel and concrete components shall be governed by the provisions of IRC:24 "Standard Specifications and Code of Practice for Road Bridges, Steel Road Bridges (Limit State Method) *(Third Revision)*" and IRC:112 "Code of Practice for Concrete Road Bridges *(First Revision)*" respectively. The loading and load factors shall be as per IRC:6 "Standard Specifications and Code of Practice for Road Bridges, Section-II Loads and Load Combinations *(Seventh Revision)*".

The approximate simplified analysis and design methods specified in these guidelines are valid for typical medium span composite box girder bridges normally adopted in our country. These are generally up to 75m in span, moderately curved and skew is restricted to 25°. Adequate number of intermediate cross frames ensures that the effect of distortion is minimal. These are fabricated from plates of width up to 2.5m and the width of the fabricated piece is also restricted to 3m for transportation. Thus, the requirement of stiffening web or flange plates is minimal.

Other types of composite box girder superstructures like with composite bottom flange, composite truss or wide box girders are excluded from the scope of this guidelines and specialist literature, including those cited in the Reference Section, may be consulted for design and detailing of such superstructures.

In the purview of this guideline, a curved box girder means it is formed using profile cutting of flanges. The stability of such girders shall be given due considerations during construction.

The following clauses of IRC:24 "Standard Specifications and Code of Practice for Road Bridges, Steel Road Bridges (Limit State Method) *(Third Revision)*" and IRC:22 "Standard Specifications and Code of Practice for Road Bridges, Section VI – Composite Construction (Limit State Design) *(Third Revision)*" shall specifically be referred to and generally followed, while designing Composite Box Girder

A) IRC:24 "Standard Specifications and Code of Practice for Road Bridges, Steel Road Bridges (Limit State Method) (*Third Revision*)"

1) 502 Materials and Properties

2) 503 Limit State Design

3) 506 Design of Tension Members

4) 507 Design of Compression Members

5) 508 Design of Truss or Open-Web Girders

6) 511 Fatigue

7) 512 Connections

8) 513 Fabrication and Inspection

9) 514 Transportation, Handling and Erection

10) Annex D Durability

11) Annex E Post-Construction Inspection and Preventive Maintenance Guidelines

12) Annex F Design assisted by Testing

B) IRC:22 "Standard Specifications and Code of Practice for Road Bridges, Section VI – Composite Construction (Limit State Design) (*Third Revision*)"

1) 606 Shear Connectors

2) 612 Fire Resistance

2) 613 Maintenance

3) Annexure-III Materials & Properties

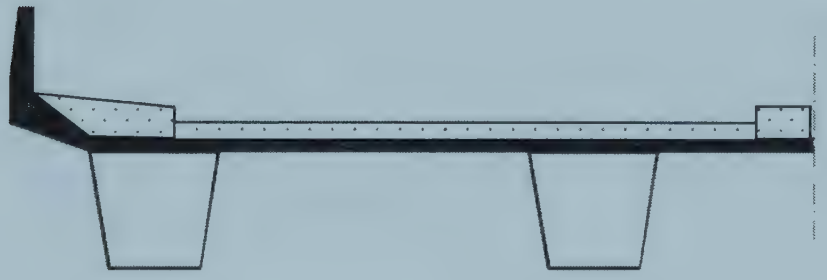
The durability requirements for internal and external surface are different. In this regard IRC:24 "Standard Specifications and Code of Practice for Road Bridges, Steel Road Bridges (Limit State Method) (*Third Revision*)" with suitable modifications as necessary may be adopted.

# 3. SALIENT FEATURES OF STEEL CONCRETE COMPOSITE BOX GIRDER SUPERSTRUCTURE

**Fig. 3.1** shows typical cross section of composite superstructure with (a) closed rectangular box girder, (b) open trapezoidal tub girder & (c) closed trapezoidal box girder. The box girders are connected by end diaphragms and intermediate diaphragms to integrate the superstructure.

(a) Composite box girder using rectangular closed steel boxes and cross girders

(b) Composite box girder using open trapezoidal steel boxes



(c) Composite box girder using closed trapezoidal steel boxes

**Fig. 3.1 Cross Sectional Arrangements**

**Fig. 3.2** shows the typical components of two single cell steel concrete box girder showing the individual components that are required to be designed and detailed.

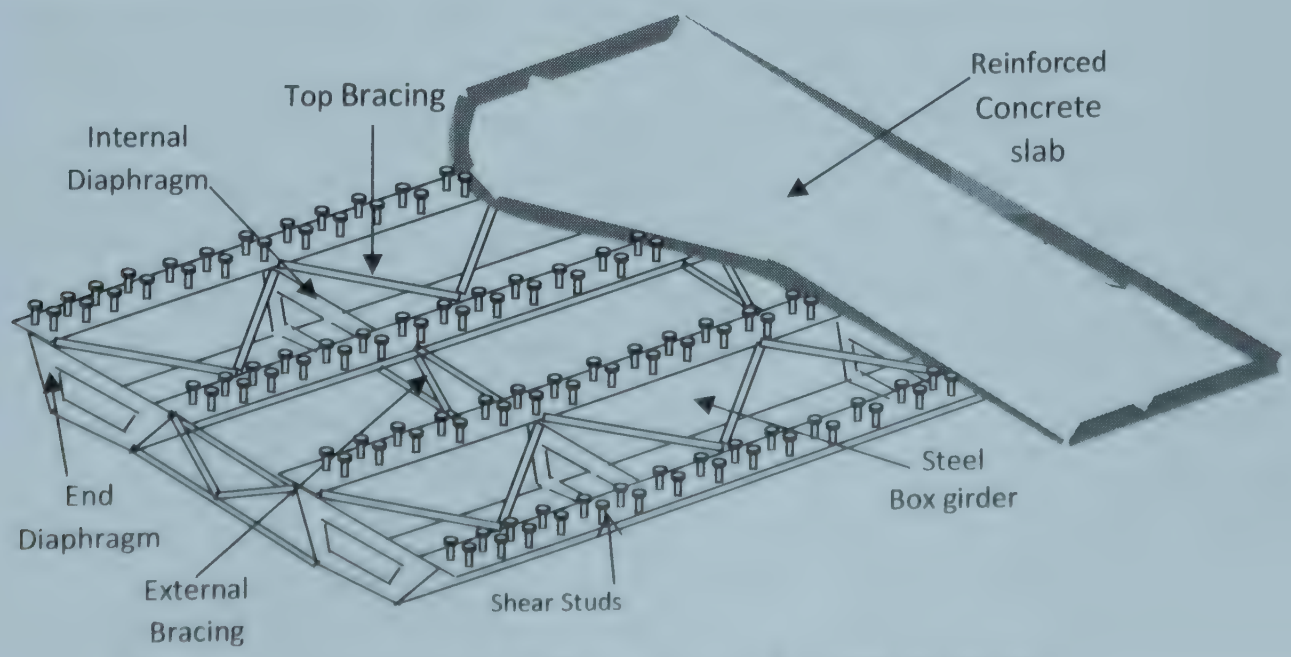



**Fig. 3.2 Components of Box Girder Superstructure**

Box girder cross section consists of two webs, connecting top and bottom flanges. Although the webs are inclined to form a trapezoidal section with larger top flange, webs normal to the flanges also can be used. The webs are made of steel plates, often stiffened (vertically and horizontally) to improve the buckling strength under shear and non-uniform compression due to flexure. The top flange is usually made of reinforced concrete to support the vehicle loading. Bottom flange is steel plate, which may be stiffened against compression due to stress reversal. The web and flange stiffeners are usually provided inside the box, so as to improve the aesthetics. Usually, the concrete at the top is cast at site after erecting the steel section (the tub). Without top reinforced concrete flange, the lateral buckling resistance of the top flange and torsional resistance of the girder are reduced. In order to enhance the stiffness and resistances before concrete hardens, bracings are provided close to the top flange plane, without interfering with subsequent concreting activity of the deck.

Bracings or transverse diaphragms are provided along the span, at the ends and at intermediate sections, to improve resistance to cross section distortion as well as transfer concentrated loads. For bracings and secondary members, generally a separate transverse analysis is carried out to ascertain the loads in these elements. Since chances of stress reversal under wind/seismic loads is there in these elements, the eccentricity in these elements should not exceed the limits specified in the codes. The end diaphragm is one of the most important structural elements of the box girder. Its design depends on its configuration. Generally, this diaphragm is a complete transverse plate, stiffened against buckling, providing a strong and stiff element against longitudinal torsion. Further it may be designed to resist the load due to superstructure lifting during bearing replacement. In such cases it needs to be designed considering plate buckling as well. In case of multiple box sections supporting a wide deck, adjacent box sections are connected with vertical bracings in the plane of end and intermediate diaphragms, as required so as to minimize differential deflection and improve torsional stiffness.

### 3.1 Behaviour of Box Girder Superstructure

In I-beam girders, global analysis determines the bending moments and shears in the main beams due to the applied loading. However, when box girders are used, additional effects such as torsion, distortion, transverse bending and shear lag are encountered as discussed below.

When a box girder is subjected to gravity load, acting in the vertical direction on the deck, it can be resolved as forces acting at the flange web junction and couples causing transverse bending **Fig. 3.3 (a).** Such a vertical load at the flange web junction can be converted into symmetric load causing longitudinal bending and asymmetric load **Fig. 3.3 (b)**. The asymmetric load on flange web junction along the web can be resolved as forces that cause distortion of the cross section **Fig. 3.3 (c)** and those that cause torsion **Fig. 3.3 (d)**. Due to wide flanges, the plane section remaining plane in longitudinal bending is not true and due to the shear lag, the bending stresses are non-uniform **Fig. 3.3 (e).**

The following structural behaviours need to be considered in design of components of box girder bridges:

- Longitudinal bending
- Shear-lag
- Transverse bending

- Shear
- Shear Connectors (partial and full)
- Saint Venant's Torsion
- Distortion
- Torsional Warping
- Distortional Warping
- Transverse Bending Due to distortion
- Interaction of Flexure, Shear, Torsion and Distortion
- Local Buckling Effects
- Cross Frame Diaphragm Behaviour
  - External
  - Internal
  - Stiffeners
    - Transverse Web Stiffeners
    - Longitudinal Web Stiffeners
    - Longitudinal Flange Stiffeners
- Fatigue
  - Shear Studs
  - Section and Joints

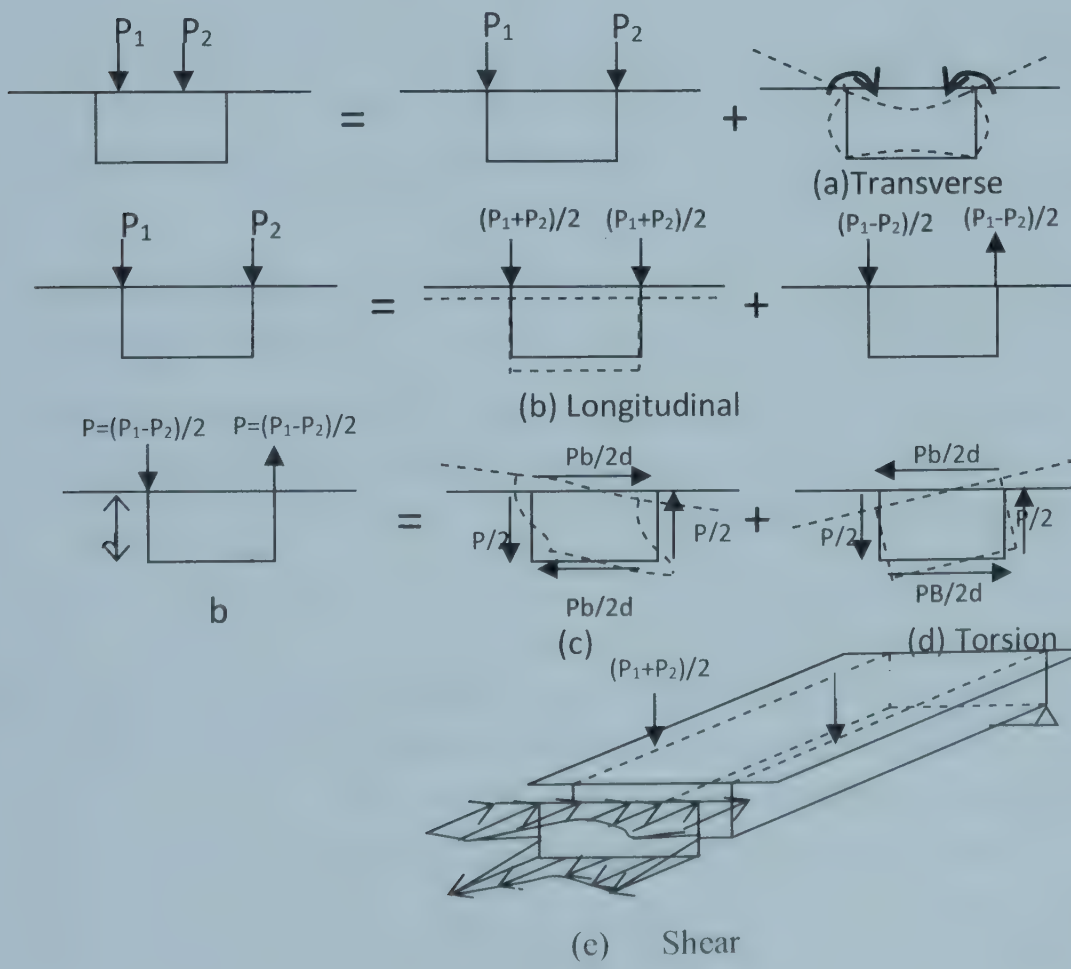



**Fig. 3.3 Structural Behavior of a Box Girder**

# 4. DIMENSIONING GUIDELINES

Appropriate care taken during preliminary design stage, considering design, detailing and construction issues, would result in an economical superstructure with ease of design, detailing and construction. Simplified approximate analysis and design methods may be followed when the dimensioning guidelines for different elements stated hereinafter are generally adhered to.

## 4.1 Span to Depth Ratio

For road bridges, the span-to-depth ratio for the composite box girder bridges will normally be around 25 for girders in simply supported spans and around 35 in continuous span girders. It is possible to further reduce the depth, if necessary, without violating deflection limitations, at the expense of additional steel. Usually a minimum girder depth of 1.5 m is maintained from the point of view of inspection and maintenance.

## 4.2 Minimum Width and Spacing

The width of the bottom flange in each girder is dictated by the overall roadway width, number of girders and viability of transportation of fabricated pieces. A minimum width of 1.2 m is usually maintained from the point of view of ease of fabrication and maintenance. The maximum width of the girder is dictated by the following functional considerations:

- Very wide bottom flange would likely require longitudinal and transverse stiffeners particularly for continuous spans. Normally plates are available of width upto 2.5 m and wider flange would call for continuous butt welding.
- Very wide deck slab due to wide spacing between webs and spacing between girders will result in thicker and hence heavier deck slab.
- Very wide spacing between webs (more than 3.6 m) may cause transportation difficulty, requiring longitudinal splicing of bottom flange at site, in shop fabricated half girders.

The ratio of the width of a girder between webs to the centre to centre spacing between girders is usually kept in the range of 1.5 to 2, unless rigorous analysis is carried out to determine the live load (wheel load) distribution between girders.

## 4.3 Bottom Flange

In continuous box girders, the bottom flange undergoes highest compressive stress due to flexure at supports. In case of wide bottom flange subjected to compression, longitudinal stiffeners may also be used, if the b/t ratio is large. Wide bottom flange subjected to tensile stresses may suffer distortion during fabrication by welding, if the b/t ratio is large. The bottom flange thickness should not be less than 12 mm and b/t ratio should not exceed 80. In extremely wide and/or slender bottom flanges, transverse stiffener may be required to provide transverse support to longitudinal stiffener, to improve the buckling strength of bottom flange under compression and torsional shear stresses. However, care should be exercised before introducing the stiffeners considering cost, constructability and poor fatigue detailing issues.

### 4.4 Webs

Straight box girder bridges experience torsion due to unsymmetrical load and skew spans, whereas horizontally curved box girders experience torsion even under symmetric loading. The webs in box girders experience flexural shear stress and torsional shear stress. Box sections are generally efficient in resisting torsion. At the construction stage, before deck concrete hardens the closed box section is not mobilized to resist torsion. Stresses due to torsion under construction of the open section should be investigated and temporary or permanent top flange level bracing should be used to improve torsional performance of the girder before concrete hardens. The shear stress, q, due to torsion can be calculated as, $q = T/2A$, where T is the torsion at the section and A is the area enclosed by webs and flanges of the box section. Due to the inclined webs in trapezoidal box girders, the component of flexural shear along the web is increased and the inclined height of the web is to be considered in the web design shear strength calculations. The slope of the web to vertical should be restricted to maximum $20^{o}$.

The webs may require transverse and longitudinal stiffeners to resist shear and flexural stresses. Experiences have shown that more economical design can be obtained by using fewer longitudinal stiffeners instead of using a thinner web with extensive transverse (and longitudinal) stiffeners or a thicker web without any stiffener.

### 4.5 Steel Top Flanges

The steel top flange of the box-girder provides connection to reinforced concrete deck through shear studs, to mobilise composite girder action. The top flange is primarily designed to carry longitudinal flexural stresses due to overall bending. Further the flanges also serve as chord members of the horizontal truss system created by the bracings at the top flange level, provided particularly to handle torsional loads and improve lateral buckling strength of the steel compression flange at the construction stage.

The top flanges also experience in-plane lateral bending stresses due to sloping webs, temporary supporting bracket for slab overhang, forces from lateral bracing system at the top flange level and horizontal girder curvature in curved bridges. The lateral bending stresses in the top flange may greatly affect the capacity of top flange to resist longitudinal bending. Increasing the top flange width is more effective than increasing the top flange thickness, to resist the in-plane lateral bending stress and improving the lateral buckling strength under longitudinal bending at the construction stage before concrete hardens. Sufficient top flange width is also required to accommodate the fasteners between top flange bracings and the top flange. However maximum width to thickness ratio provisions should be kept in mind while deciding on the width and thickness of steel top flange.

Top flanges have to be checked for erection loads, especially local stresses in the top flange and weld between top flange and web due to the erection loads.

### 4.6 Internal Intermediate Diaphragms

Torsion of the box section that do not act in the same pattern as the St Venant's Torsional flow in the cross section causes distortion of the cross section **Fig. 3.3 (c)**. The internal intermediate

diaphragms are provided to control the cross-sectional distortions and minimize associated undesirable additional stresses.

The spacing of the internal diaphragms is a part of the framing process. They are normally provided at the sections where internal vertical stiffeners are present. Such vertical stiffeners should be connected to the top flange. The top flange bracings also normally intersect at the location of the interior diaphragms. They normally have K configuration bracings to provide better access for construction, inspection and maintenance. X or Z configuration may be used in narrow box sections. Strength of bracing is normally controlled by bracing forces at the maximum moment section. Bracing forces could be quite large in curved box girder bridges with relatively larger spacing. Handling forces may sometimes govern the bracing design requirement.

### 4.7 External Intermediate Diaphragms

The external intermediate diaphragms are provided to tie the box girders together to minimize differential deformations. Such differential deformation, in the absence of the intermediate external diaphragms could cause fit-up problem before concrete slab placement during erection and differential slab thickness. During service these external intermediate diaphragms reduce the bending stresses due to differential deformation of the box girders.

The external intermediate diaphragms present fatigue concerns, since connection of the diaphragm bottom chord to the bottom flange is problematic and connection to the web vertical stiffener may result in poor fatigue detail.

### 4.8 Internal Diaphragms at Supports

Internal diaphragms at piers and end supports are generally full depth plate girder sections with access hole in the web. These diaphragms may be supported by one or two bearings. Two supports resist torsion at the supports. Single bearing is often recommended due to the width limitations of the bottom flange and demand for high construction accuracy to place all bearings at appropriate elevation.

The diaphragm is designed as deep beam for the bending loads due the shear from the webs (in case of single bearing support and double bearings not located at the web flange junction) and shear flow along the circumference due to the torsional moment reaction on the box girder. The diaphragm typically consists of a vertical plate serving as a web. Top flange is provided to the deep beam to increase the bending strength and also to provide support to the slab at girder ends. The end bearing stiffeners are attached to the diaphragms and are designed as a column to resist a load equal to the reaction. The access holes in the diaphragm should be detailed considering the flow of stresses due to bending, shear due to flexure and torsion.

## 4.9 External Diaphragms at Supports

In case double bearings are used and the individual girders are properly anchored to adequately carry the torsional reaction, external diaphragms at supports connecting adjacent box girder may not be necessary. However, if a single bearing is provided at each box girder, then the end diaphragm should be designed as a deep girder to transfer shear and moment associated with end torsional reaction of the bridge super structure **(Fig. 4.1)**. Such torsional reaction could be high in curved span bridges.

Generally, suitably designed openings are provided in the diaphragms to access the inside of the box from pier/ abutment cap. Care should be taken to ensure that the opening will be accessible. In case of continuous girders, the access hole has to be suitably located in the bottom flange.

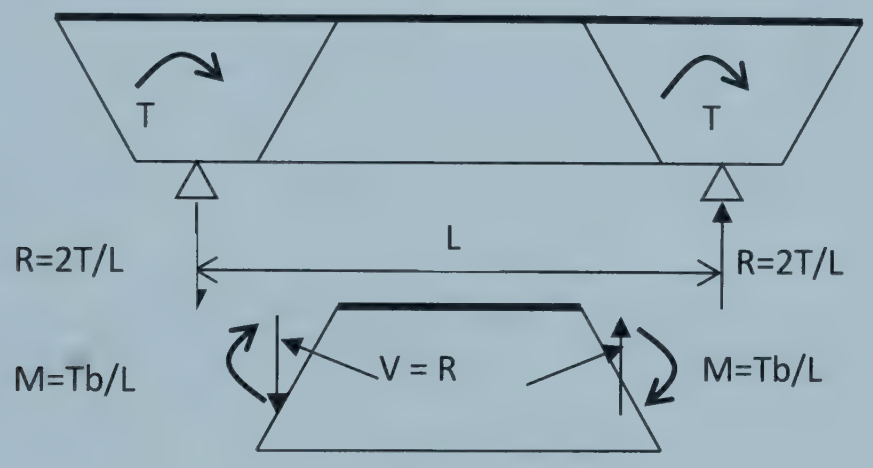



**Fig. 4.1 External End Diaphragm Forces**

## 4.10 Top Flange Lateral Bracing

Until the top concrete slab hardens, the tub type box girder behaves as an open section with poor torsional stiffness and laterally unsupported top flange under compression. In order to improve these properties top flange lateral bracings are provided with top flange forming a part of the lateral bracing system. The diagonals of the bracing system are typically made of angle or T Sections. The top lateral bracing may be temporary until the concrete hardens or permanently left in place. The angle of the diagonals with the longitudinal axis should be preferably 45° but should be between $30^0$ to $60^0$. The diagonals are usually framed into the top flange at the intersection of the top flange and the internal diaphragms. If, in addition to diagonals, bracing members perpendicular to the longitudinal axis of the box girder is introduced to form K or N configuration, it makes the structure stiffer and increases the bracing force, since overall flexure on the box section causes forces in the bracings, due to shortening of the top chord to which the bracings are attached. Further due to torsion before concrete hardens, there is shear in the top bracings corresponding to the shear in the top flange plane based on St Venant's Torsional Shear. Vertical loads on the top flange also introduce brace forces due to sloped webs, which is mostly resisted by lateral strut between the top flanges. External diaphragms between the twin box girders also induce forces in the top flange bracing system.

# 5. METHODS OF ANALYSIS

The box girder is essentially a thin-walled structure and analysis procedure applied should be able to estimate the effects of torsion and distortion besides the effects due to longitudinal bending. The internal forces and stresses are calculated by any one of the methods available on different mathematical models of the box-girder superstructure. There are many different methods of analysis of Box-Girder Superstructures, depending upon the level of complexity one wants to deal with, simplifying assumptions and structural actions one wants to consider and access to computational facility. Different types of structural actions can be considered individually under a given loading and the results may be superimposed to give critical design values, in the case of loading under the linear elastic range of behaviour, based on the method of superposition. It is also possible to obtain the behaviour under combined loading, considering the non-linear behaviour using the finite element method. The different methods of analysis and their relative merits are discussed briefly in the following sections. Other acceptable methods can also be followed.

A global analysis is carried out to determine the maximum forces and moments at the critical parts of the bridge, under the variety of possible loading conditions. Local analysis of the deck slab is usually carried out separately from the global analysis for design of the reinforcement.

For the type of box girders considered in this guidelines, there will normally be sufficient intermediate cross-frames or diaphragms. That arrangement will restrain distortional effects and ensure that simple global analysis will be adequate. A more detailed analysis may be needed, involving the use of finite element programs, for box girders that are provided with very little distortional restraint. Normally restraints at a spacing of about three times the depth of the girder will limit the reduction in effective stiffness to a level which can be neglected in global analysis.

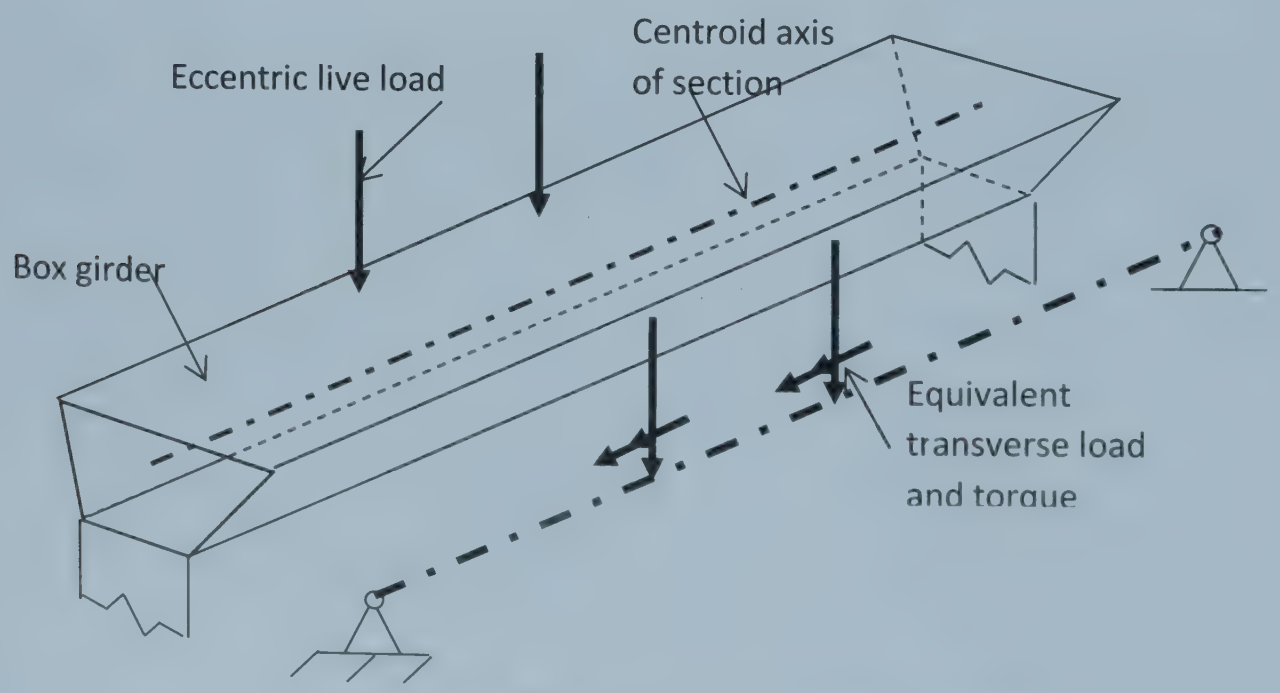



**Fig. 5.1 Simple Beam Model**

## 5.1 Simple Beam Theory

The composite box girder is modelled as a simple linear beam along the centroid of the cross section. The live load is applied as concentrated or uniformly distributed load, as desired. The eccentricity of live load in the vertical direction transverse to the beam can be considered

through a combination of concentric vertical load through the centroid of the beam model and torque with respect to centroid **(Fig. 5.1)**.

Only bending moments, torsional moment and shear force on sections normal to the longitudinal axis are obtained from the simple beam model. All the other effects are not accounted for in the simple beam model. They have to be separately accounted for by using other simplified methods, available to specifically evaluate them.

Although a single beam model may be suitable for analysing narrow single box span or during construction stage, Grillage Analysis is suitable for multi-girder spans during service as well as construction stages.

## 5.2 Grillage Analysis

The basis of most commonly used computer models for I-beam and slab bridges is the grillage analogy. In this analogy the structure is idealized as several longitudinal and transverse beam elements in a single plane, rigidly interconnected at nodes. Transverse beams may be orthogonal or skewed with respect to the longitudinal beams **(Fig. 5.2)**.

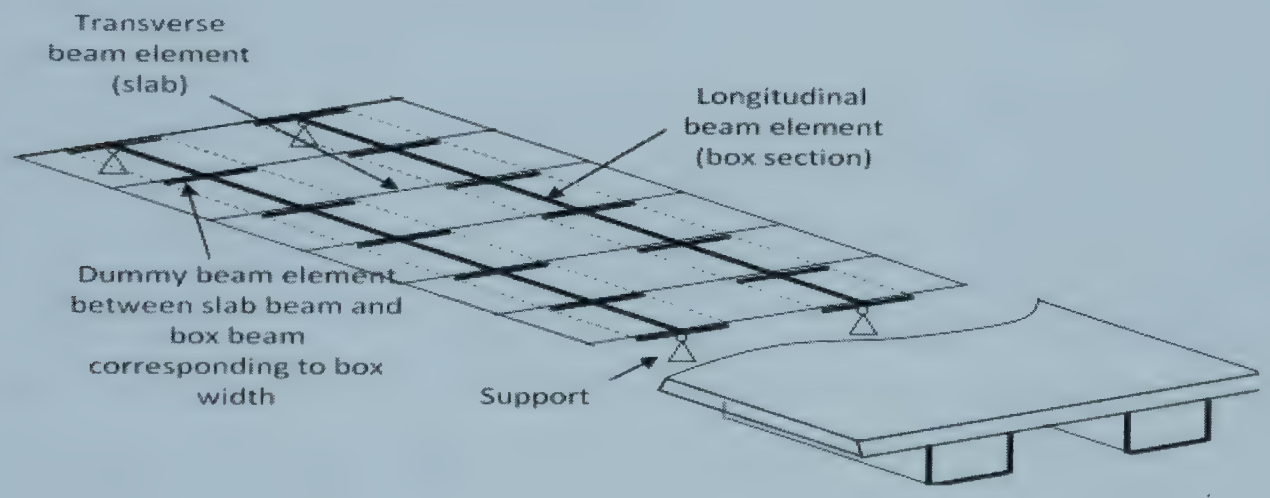

Transverse beam element (slab)
Longitudinal beam element (box section)
Dummy beam element between slab beam and box beam corresponding to box width
Support

**Fig. 5.2 Grillage Model**

Provided that distortional effects are not significant, composite box girders can also be analysed by this method. The box girder is modelled as a series of beam elements along its centreline. Its section properties (bending as well as torsional stiffness) are based on the construction stage. The slab is represented by a series of transverse beams. Stiff dummy elements representing the width of the box girder are used to connect the slab elements with the box girder elements. Cross bracing between box girder, wherever provided, should also be modelled. For service stage analysis, the properties of the girder elements shall be computed on the basis of composite section. Torsional properties should be estimated on the basis of uncracked section except at the support sections of continuous spans. Appropriate sectional properties should be used for construction stage(s) analysis.

The effect of inadequate distortional support of a box section effects in reduction of torsional stiffness. Intermediate restraints lead to substantial reductions in the distortional deflections. Generally, restraints at a spacing of about three times the depth of the girder usually limit the reduction in effective stiffness to a level which can be neglected in global analysis. Distortion of open sections during construction stage also needs to be considered carefully since an open

section is weak in torsion. Temporary bracing of top flange or forming the slab at discreet locations may be considered. The grillage analysis method naturally cannot be applied to analysis of tub girder during construction stage. Separate analysis is necessary to size the top flange bracing system.

The spacing and width of the main longitudinal beams control the transverse node spacing. The spacing of transverse beams (representing the slab) should not exceed about 1/8 of the span. Section properties for these elements representing the slab alone should use a width equal to the element spacing. Torsional stiffness of the slab should be equally divided between the transverse and longitudinal beams. Section properties for transverse beam elements representing transverse bracing or cross girders should be determined based on both the bending stiffness and the shear stiffness of the members acting with the deck slab.

It is to be noted that the grillage analysis does not directly account for warping and distortional effects, transverse bending effects and the effects of shear lag. These have to evaluated separately and considered in design.

### 5.3 Finite Element Analysis (FEA)

In the finite element method, the entire structure is modelled using finite number of flat shells or solid elements corresponding to top flange, webs and bottom flange and beam elements, shell elements or solid elements corresponding to stiffeners. The bracings may be modelled using truss element. On application of constitutive relationship of the elements, their assembly and the boundary conditions, one obtains the set of equations representing the equilibrium of the structure, solution of which yields the displacements and internal stresses. With appropriate model the finite element method accounts for the effect of shear lag, local effects of wheel loads on slabs, effects of transverse bending, effects of torsions, distortion and warping. The detail of the effects which can be evaluated, and the accuracy depends on the fineness of the mesh and the capabilities of the elements used. FEM can be used with different degrees of sophistication for different purposes. Most common is linear elastic analysis where the material and the system are assumed to behave linearly. Modelling is very important in order to arrive at proper solution. The size of the element (fineness of meshing) affects the size of the problem. A too coarse mesh may lead to unconservative results when local buckling governs. Successive finer meshing is often carried out until stable converged results are achieved.

### 5.4 Torsion, Distortion and Warping

Global analysis of beam and slab span by grillage method of analysis determines the bending moments and shears in the main beams due to the applied loading and those are used to design the beam section. Two additional effects, namely torsion and distortion, have to be considered for design of box girders. Torsional effects are determined by the global analysis. Distortional effects are limited to local regions between intermediate diaphragms.

#### 5.4.1 *Torsion and Distortion*

The general case of an eccentric load applied to a box girder is in effect a combination of three components - bending, torsion and distortion. As a first step, the force can be separated into two components, a pair of symmetric vertical loads (causing flexure) and a couple (causing

torsion), as shown in **Fig. 5.3(a)** However, torsion is in fact resisted in a box section by a shear flow around the whole perimeter. Hence, the couple should in turn be separated into two parts, representing pure torsion and distortion, as shown in **Fig. 5.3(b)**.

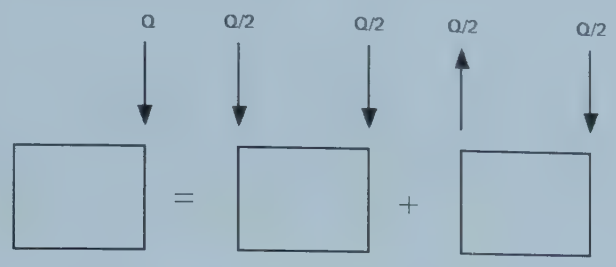



**Fig. 5.3(a) Separation of an Eccentrically Applied Load into Two Components**

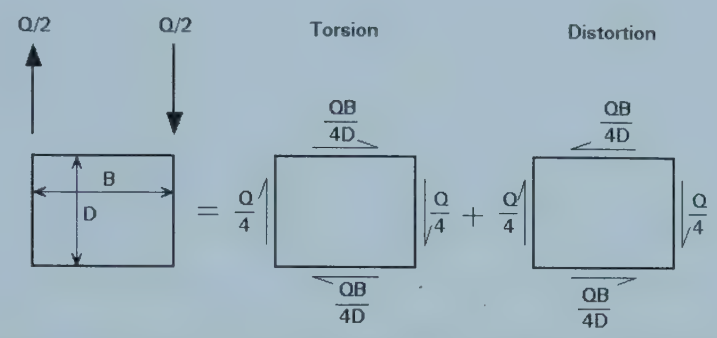



**Fig. 5.3(b) Separation of Force Couple into Torsion and Distortion Components**

The first two components, vertical bending loads and a torsional shear flow, are externally applied forces, and they must be resisted in turn at the supports or bearings. The third component, distortional forces, comprises an internal set of forces, internally statically in equilibrium and do not give rise to any external reaction. Distortional effects depend on the behaviour of the structure between the point of application and the nearest positions where the box section is restrained against distortion.

### **5.4.2** *Torsion and Torsional Warping*

The theoretical behaviour of a thin-walled box section subject to pure torsion is described in standard texts. For a single cell box, the torque is resisted by a shear flow which acts around the walls of the box. This constant shear flow (force/unit length) around the box is given by q = T/2A, where T is the torque and A is the area enclosed by the centre line of box wall thickness. The shear flow produces shear stresses and strains in the walls and gives rise to a twist per unit length, which is given by the general St. Venant's Torsion Expression

$$\theta = \frac{T}{4A^2G}\int\frac{ds}{t} \text{ or } \theta = \frac{T}{GJ}$$

Where J is the torsion constant and A = area enclosed by the centre lines of the wall thickness.

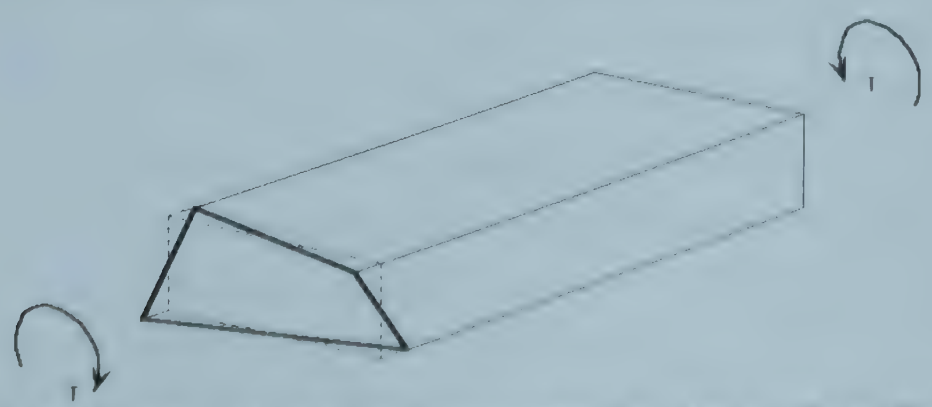



**Fig. 5.4 Warping of a Rectangular Box Subject to Pure Torsion**

However, this pure torsion of a thin-walled section also produces warping of non-symmetric cross-section. As shown in **Fig. 5.4** for a rectangular box section, that is free to warp at its ends. However, in practice boxes are not subject to pure torsion. Wherever there is a change of torque (at a point of application of load or at a torsional restraint) there is restraint to warping, because the 'free' warping displacements due to the different torques would be different. Such warping restraint gives rise to longitudinal warping stresses and associated shear stresses in each wall of the box section.

### 5.4.3 *Distortion*

When torsion is applied directly around the perimeter of a box section, by forces exactly equal to the shear flow in each of the sides of the box, there is no tendency for the cross section to change its shape. Torsion can be applied in this manner if, at the position where the force couple is applied, a diaphragm or stiff frame is provided to ensure that the section retains its shape. In this case the torque is in fact fed into the box walls as a shear flow around the perimeter.

Provision of such diaphragms or frames is practical, at supports and at positions where heavy point loads are introduced. But such restraint can only be provided at discrete positions. When the load is distributed along the beam, or when eccentric point loads can occur anywhere along the beam such as the concentrated axle loads from vehicles, the distortional effects must be carried by other means.

The distortional effects shown in **Fig. 5.5** tend to increase the length of one diagonal and shorten the other. This tendency is resisted in two ways, namely, by in-plane bending of each of the wall of the box and by out-of-plane bending of the box as the frame.

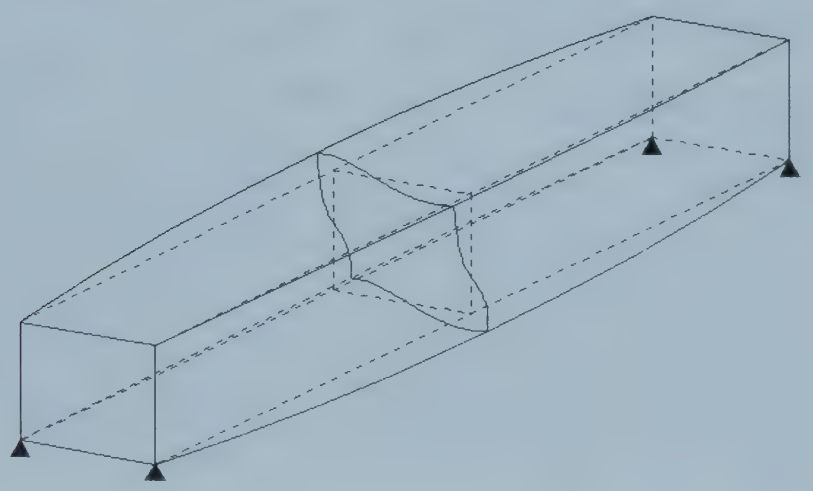


**Fig. 5.5 Distortional Displacements in a Box Girder**

The distortional behaviour depends on interaction between these two forms of bending. The behaviour has been demonstrated to be analogous to that of a beam on an elastic foundation (BEF), and this analogy is frequently used to evaluate the distortional effects.

The distortional deflections in most situations would be significant in absence of intermediate cross-frames to restrain distortion and would affect the global behaviour. Hence it is usual to provide several intermediate cross-frames or diaphragms. Consideration of the distortional displacements and stresses is then limited to the lengths between such cross-frames.

# 6. LONGITUDINAL BENDING

## 6.1 General

Longitudinal bending analysis of closed steel box composite structure or steel tub composite structure shall be checked for two stages, namely construction stage and service stage.

During construction stage RCC deck is not hardened & bare steel structure carries the load if unsupported condition of construction is considered. Stresses during construction may well govern the design of some elements, such as the top flange. The designer should consider at least one feasible construction sequence and ensure that the strengths are adequate at critical locations at every stage of construction. It is usual for the deck slab to be concreted in stages and for the steel girders to be unpropped between supports during this process. Part of the load is thus carried by the steel beam sections alone, part by the composite sections. Several separate analyses are therefore required, each representing a different stage. Typically, there are about twice as many stages as spans, since concrete is usually placed alternately in midspan regions and over supports. Where the cantilevers are concreted at a different stage from the main width of slab, this must be considered in the analyses. When an open box is concreted in stages, there will inevitably be stages when parts of the beams are closed sections and parts are open sections. In such staged construction concrete may well be placed over the supports before the span regions, to develop an effective top flange at an early stage. Distortional effects should be considered carefully when concreting open sections in stages.

During service stage behaviour when the deck is hardened & composite behaviour is considered.

## 6.2 Effective Width Models for Global Analysis

The effects of shear lag and of plate buckling on the stiffness of members and joints shall be taken into account in the global analysis.

The effects of shear lag of flanges in global analysis may be taken into account by the use of an effective width. Refer **Clause 603.2** of IRC:22 "Standard Specifications and Code of Practice for Road Bridges, Section VI – Composite Construction (Limit State Design) Third Revision" for further details.

The effects of plate buckling in elastic global analysis may be taken into account by effective cross-sectional areas of the elements in compression. Refer **Clause 503.6** & **503.7** of IRC:24 "Standard Specifications and Code of Practice for Road Bridges, Steel Road Bridges (Limit State Method) (Third Revision)" for further details.

For global analysis the effect of plate buckling on the stiffness may be ignored when the effective cross-sectional area of an element in compression is larger than 0.5 times the gross cross-sectional area of the same element.

### 6.3 Effective Cross Section

In calculating longitudinal stresses, account should be taken of the combined effect of shear lag and plate buckling.

The effective cross-sectional properties of members should be based on the effective areas of the compression elements and on the effective area of the tension elements due to shear lag. **(Fig. 6.1)**.

The effective area $A_{eff}$ should be determined assuming that the cross section is subject only to stresses due to uniform axial compression. For non-symmetrical cross sections, the possible shift $e_N$ of the centroid of the effective area $A_{eff}$ relative to the centre of gravity of the gross cross-section, Refer **Fig. 6.2**, gives an additional moment which should be taken into account in the cross-section verification.

The effective section modulus should be determined assuming the cross section is subject only to bending stresses, Refer **Fig. 6.3**. For biaxial bending effective section moduli should be determined about both main axes.

The stress in a flange should be calculated using the elastic section modulus with reference to the mid- plane of the flange.

Additionally, refer to **Clause 603.2.3** of IRC:22.

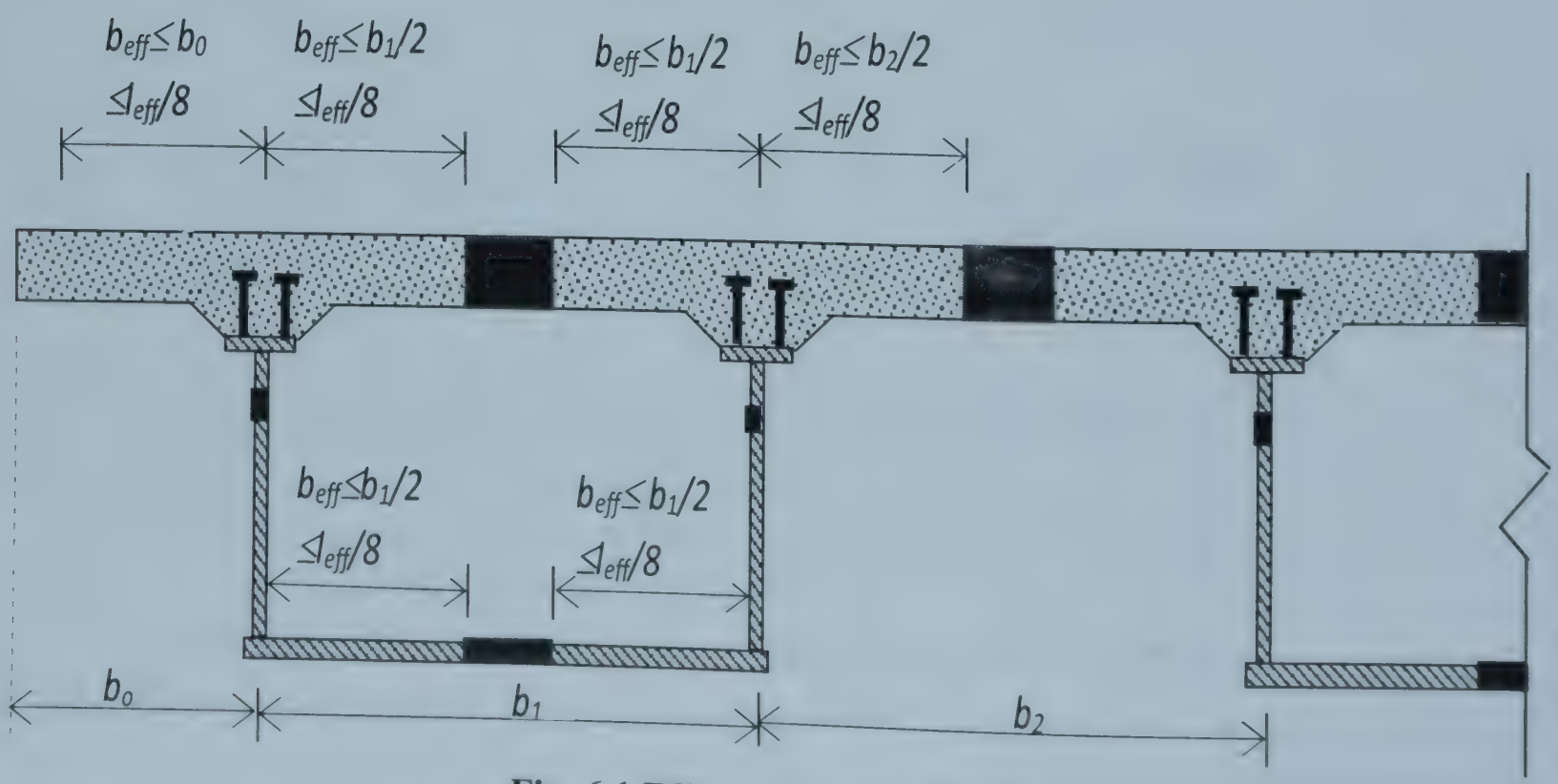



**Fig. 6.1 Effective Cross Section**

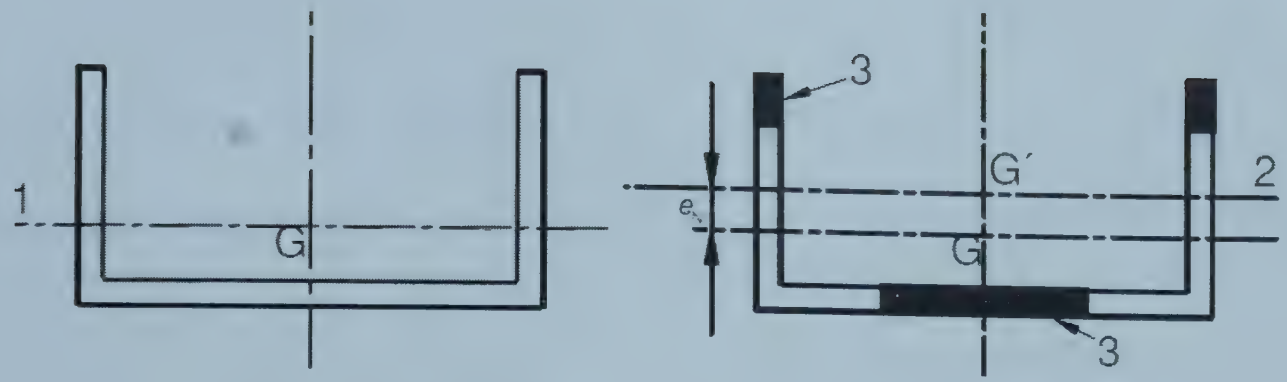



**Fig. 6.2 Class 4 Cross Sections - Axial Force**

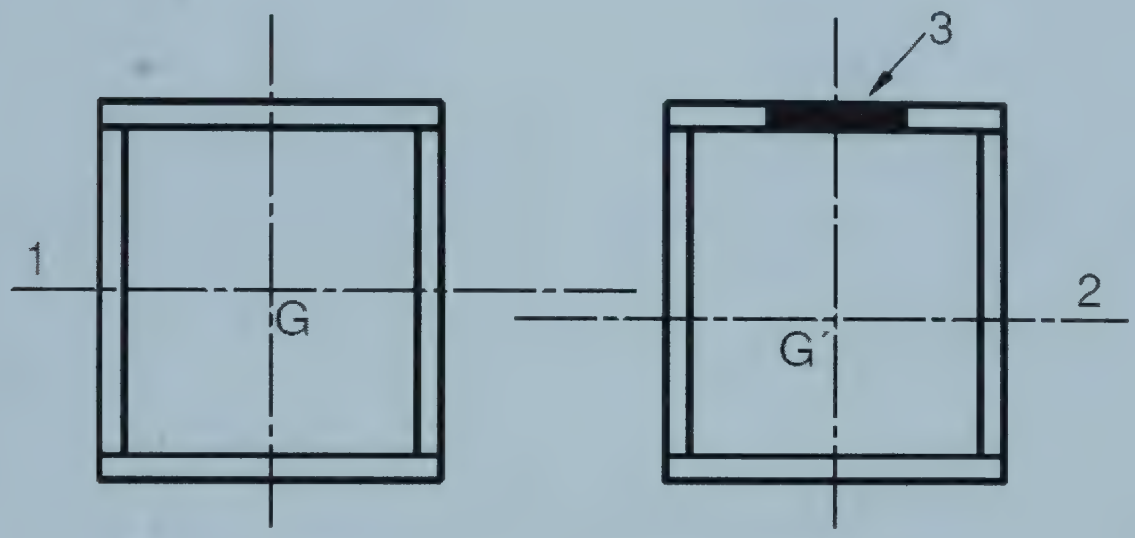



**Fig. 6.3 Class 4 Cross Section - Bending Moment**

where,

G centroid of the gross cross section,

G′ centroid of the effective cross section,

1 Centroidal Axis of the gross cross section,

2 Centroidal Axis of the effective cross section

3 Ineffective zone

## 6.4 Effective Span

Refer **Fig. 6(b)** of IRC:22.

## 6.5 Compression Flange Buckling Requirement

Refer **Clause 509.6.1.2** of IRC: 24.

## 6.6 Lateral Torsional Buckling of Box Girder

The value of $\lambda_{LT}$ for overall lateral buckling of a beam of rectangular or trapezoidal box section, uniform within the half-wavelength of buckling of the compression flange, should be taken as:

$$\lambda_{LT} = 2.25\,\eta\,\xi\,\sqrt{\frac{Z_{pe}\,l_e}{r_y\sqrt{A\,J}}}$$

where,

$\eta$ = factor accounting for bending moment variation within the half-wavelength of buckling of the compression flange and usually taken as 1.0

$Z_{pe}$ = Plastic Modulus of the Effective Section

$l_e$ = Effective length for Lateral Torsional Buckling, refer **Clause 509.3** of IRC:24

$r_y$ = Radius of Gyration of the Gross Cross-Section of the beam about its Y-Y axis

A = area of the Gross Cross-Section

$J$ = Torsional Constant = $\frac{4 A_0^2}{\Sigma(B / t)}$

$A_0$ = Area enclosed by the Median Line of the Perimeter of the Section

B, t = Width and Thickness of each wall of the section forming the closed Perimeter

$$\xi = \left(\frac{(I_x - I_y)(I_x - 0.385J)}{I_x^2}\right)^{0.25}$$

$I_x$, $I_y$ = Moment of inertia about axes through the centroid, normal to the plane of bending and in the plane of bending, respectively

The limiting moment of resistance, $M_R$, should be obtained from **Fig. 6.4(a)** for beams fabricated by welding (excluding local welding of stiffeners) or **Fig. 6.4(b)** for all other sections according to the value of:

$$\lambda_{LT}\sqrt{\left(\frac{\sigma_{yc}}{250}\right)\left(\frac{M_{ult}}{M_{pe}}\right)}$$

where,

$M_{ult}$ = Moment of Resistance of cross section if Lateral Torsional buckling is prevented,

$M_{ult} = M_{pe}$ for Compact Sections

$M_{ult}$ = the least of $Z_{xc}\sigma_{yc}$, $Z_{xt}\sigma_{yt}$ or $Z_{xw}\sigma_{yw}$ for non-compact sections

$Z_{xc}$, $Z_{xt}$ = Elastic Moduli of the section with respect to the extreme Compression and Tension Fibres respectively

$Z_{xw}$ = Minimum Modulus of the section with respect to the web based on the effective section

$\sigma_{yc}$ = Nominal Yield Stress value for the compression flange material

$\sigma_{yt}$ = Nominal Yield Stress value for the tension flange material

$\sigma_{yw}$ = Nominal Yield Stress value for the web material

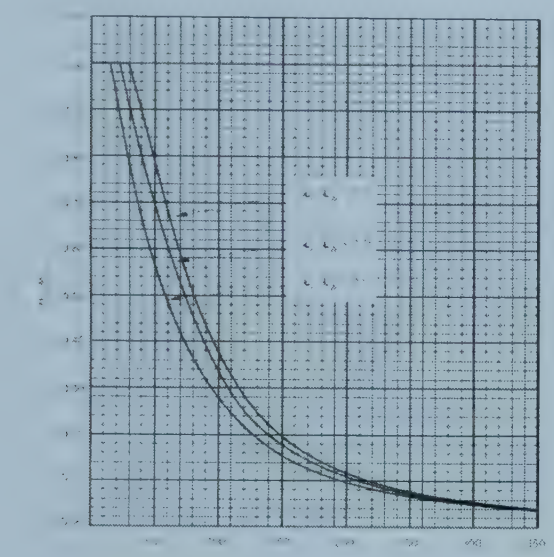
$l_w$ = Half Wavelength of buckling.

If intermediate restraints are fully effective, $l_w = l_e$.

If there are no intermediate restraints or the restraints are not fully effective, the $l_w$ = Taking $L/l_w$ as the next integer below $L/l_e$, but not less than 1.

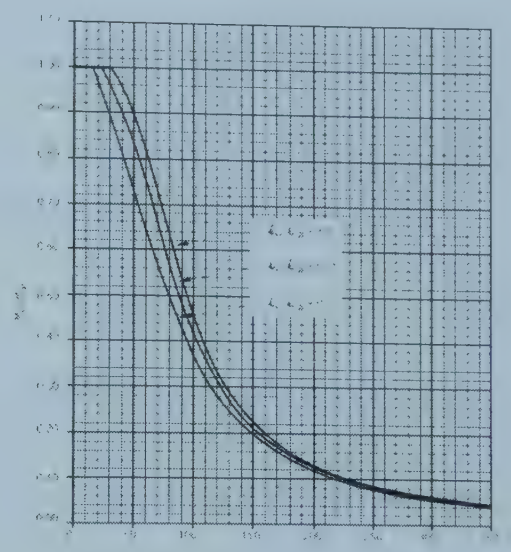
$l_e$ = Effective Length for lateral Torsional Buckling

L = Distance between the restraints at the supports of the span of the simply supported or continuous beam under consideration



$\lambda_{LT}\sqrt{\left(\frac{\sigma_{yc}}{250}\right)\left(\frac{M_{ult}}{M_{pe}}\right)}$

(a)



$\lambda_{LT}\sqrt{\left(\frac{\sigma_{yc}}{250}\right)\left(\frac{M_{ult}}{M_{pe}}\right)}$

(b)

**Fig. 6.4 Limiting Moment of Resistance $M_R$ [3]**

The bending resistance $M_d$ is given by:

$M_d = M_R / (\gamma_m \gamma_{f3})$ for compact and non-compact section

where, $\gamma_m = 1.05$ and $\gamma_{f3} = 1.1$

The mode of buckling is of importance during construction particularly for open top sections. If the section is too sensitive to a particular mode of buckling, either a wider section should be used, or plan bracing should be provided at top flange level. The plan bracings need be provided along the entire span provided an exhaustive buckling analysis is carried out. If the open box has inclined webs, buckling will be normal to the web, not in the plane of the flange.

### 6.7 Design for Ultimate Limit State

For laterally supported box girders during construction, Refer **Clause 509.2** & **509.2.1** of IRC:24.

The moment of resistance of a composite box girder can be calculated as per the **Annexure-I** of IRC:22 with due considerations to the effects of shear lag and of plate buckling. Due modifications shall be made to account for the respective geometry of the box girder.

### 6.8 Design for Serviceability Limit State

Refer **Clause 604** of IRC:22.

### 6.9 Design for Fatigue Limit State

Refer **Clause 605** of IRC:22.

# 7. SHEAR

Refer **Clause 509.4** of IRC:24.

# 8. MEMBERS SUBJECTED TO COMBINED FORCES

Refer **Clause 510** of IRC:24.

# 9. TORSION, DISTORTION AND WARPING

## 9.1 General

The traditional methods for evaluating torsional and distortional actions inbox girders are described here. With FEM analysis using 2D elements, the displacements and stresses can be estimated more readily. However, an Finite Element analysis merges the longitudinal stresses due to axial force, bending, torsional warping and distortional warping. When evolving a design, knowledge of the relative magnitudes of the separate components can be useful, and these algebraic expressions provide such knowledge.

The shapes of the theoretical stress patterns shown in this section can be useful when interpreting an Finite Element analysis although, in situations where shear-lag is particularly significant, the patterns will differ slightly from those shown here.

The use of **Clause 9.2** & **9.3** for the calculation of distortional effects requires that the distortional restraints satisfy the given stiffness criteria.

If the effects of St Venant Torsion are neglected in open sections, the full applied torsional moment should be carried as warping torsion.

## 9.2 RESTRAINT OF TORSIONAL WARPING IN BOX GIRDERS

### 9.2.1 *General*

When an increment of torque, $T$, is applied at a section of a box girder (other than at a free end) the resulting maximum longitudinal stress at this section due to restraint of torsional warping, $\sigma_{TW}$, may be calculated as follows:

### 9.2.2 *Stresses at the Junction between the Bottom Flange and Web*

At the junction between the bottom flange and a web, at the section where an increment of torque, $T$, is applied:

$\sigma_{TWB} = DT / I_T$;

where:

$D$ is the depth of the box at its centreline measured between centres of flange plates, or, in composite construction, between the effective centroid of the composite top flange and the centre of the bottom flange plate;

$I_T$ is the torsional constant $\frac{4\,A_o^2}{\Sigma\frac{b}{t}}$;

$A_o$ is the area enclosed by the median line of the perimeter material of the section;

$b$, $t$ are the width and thickness, respectively, of each wall of the section forming the closed perimeter.

For boxes with a composite flange, the St. Venant Torsional Stiffness of box girders should be calculated for a transformed cross section in which the concrete slab thickness is reduced by the modular ratio. OG = Ga/Gc where Ga and Gc are the elastic shear moduli of structural steel and concrete respectively. The effects of creep should be taken into account with appropriate modular ratio in accordance with **Clause 604.3** of IRC:22.

**9.2.3** *Stresses at the Junction between the Top Flange and the Web*

At the junction between the top flange and a web, at the section where an increment of torque $T$ is applied:

$$\sigma_{TWT} = \left(\frac{B_B}{B_T}\right)^2 \frac{DT}{I_T\left(1+\frac{2B_C}{B_T}\right)^3};$$

where:

$B_C$ is the Width of Flange Projection beyond the centre of the web;

$B_B$, $B_T$ are the Widths of the Bottom and Top flanges, respectively, measured between the centres of webs.

Where there are two or more box girders in a single structure with connected top flanges, $\sigma_{TWT}$ may be taken as zero.

**9.2.4** *Distribution of Stresses*

At a distance x from the section where an increment of torque is applied:

$$\sigma_{TW,x} = \sigma_{TW}{}^{e^{-(2x/B_B)}}$$

The distribution across the section of the longitudinal stress due to restraint of torsional warping may be assumed to be as shown in **Fig. 9.1**.

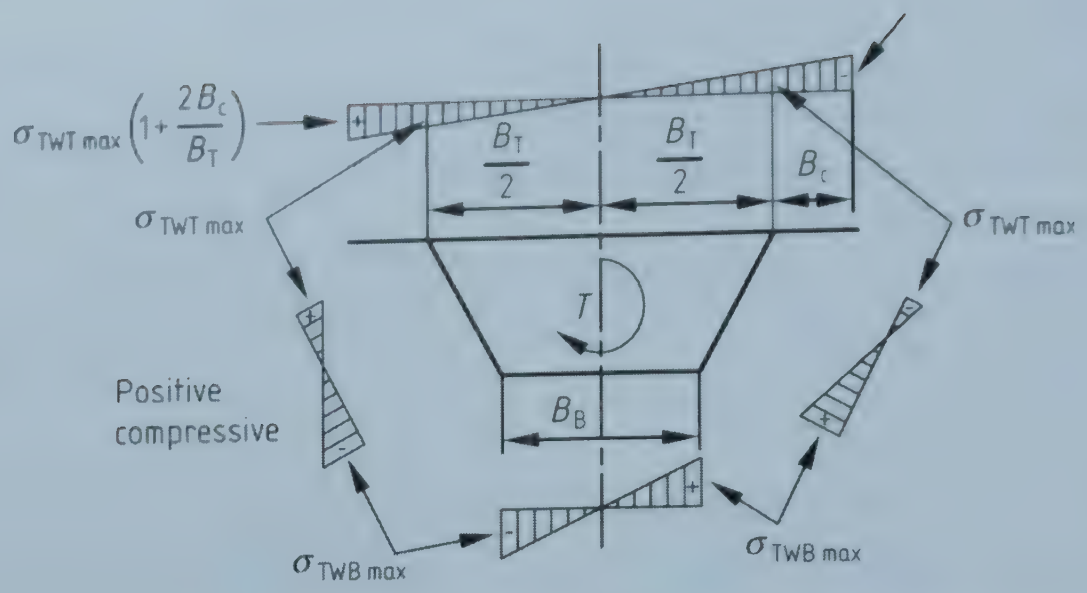



**Fig. 9.1 Distribution of Torsional Warping Stresses**

## 9.3 Restraint of Distortional Warping

### 9.3.1 *General*

When torque is applied to a box girder other than at a cross frame or a diaphragm, the resulting longitudinal stress due to restraint of distortional warping $\sigma_{DW}$ may be calculated in accordance with **Clause 9.3.2** provided that the cross frames or diaphragms are in accordance with **Clause 9.3.4**.

### 9.3.2 *Corner stresses*

The distortional warping stress $\sigma_{DW}$ may be calculated as follows.

a) At a junction between a flange and a web, under a uniformly distributed applied torque $T_{UD}$ per unit length of span:

$\sigma_{DW} = \frac{T_{UD}\bar{z}\,L_D^2}{4{,}5B_T I_{yy}}$ when $\beta L_D < 1.6$;

$\sigma_{DW} = 0{,}6\frac{T_{UD}\bar{z}\,L_D^2}{\beta^2 L_D^2 B_T I_{yy}}$ when $\beta L_D \geq 1.6$;

where:

$\bar{z}$ is the distance from the horizontal neutral axis to the flange/web junction;

$I_{yy}$ is the second moment of area of the girder, inclusive of its effective flanges, about the horizontal neutral axis;

$L_D$ is the spacing of cross frames or diaphragms;

$B_T$ is as defined in **Clause 9.2.3**;

$\beta L_D$ is given by $\left(\frac{KL_D^4}{EI_{yy}}\right)^{0.25}$;

$K$ is given by $\frac{24\,D_{YT}R_D}{B_T^3}$;

$D_{YT}$ is the transverse flexural rigidity, *EI*, of the top flange, including Transverse Stiffeners if any, per unit length of span;

$$R_D = \frac{B_B+B_T}{B_B\left(\frac{B_B}{B_B+B_T}\right)\left(\frac{2D_{YT}\ d}{D_{YC}\ B_T}+1\right)-B_B V_D\left\{\left(2+\frac{B_B}{B_T}\right)\frac{D_{YT}\ d}{D_{YC}B_T}+1\right\}};$$

where;

$V_D$ is as defined in **Clause 9.4.2**;

$B_B$, $B_T$ are as defined in **Clause 9.2.3**;

$D_{YB}$ is the transverse flexural rigidity, *EI*, of the bottom flange, including transverse stiffeners, if any, per unit length of span;

*d* is the clear depth of the web measured in the plane of the web, or, if corner stiffening is provided, the distance between the centers of connections of such stiffening to the web;

$D_{YC}$ is the transverse flexural rigidity, *EI*, of the web, including its transverse stiffeners, if any, per unit length of span.

b) At a junction between a flange and a web, due to a concentrated applied torque, T where the axle or concentrated load is applied mid-way between diaphragms:

$\sigma_{DW} = \frac{T\ \bar{z} L_D}{B_T I_{yy}}$ when $\beta L_D < 1.0$;

$\sigma_{DW} = \frac{T\ \bar{z} L_D}{\beta L_D B_T I_{yy}}$ when $\beta L_D \geq 1.0$;

c) Under uniformly distributed loading (UDL) and concentrated loading, the effects due to each of them should be separately calculated as described in a) and b), and the sum of the resulting stresses taken.

d) Under a series of concentrated torques due to axle loads:

$\sum\sigma_{DW} = \sigma_{DW1}\sum(C_{\beta x})$;

where:

$\sigma_{DW1}$ is the value of $\sigma_{DW}$ obtained under b) for a unit axle load;

$C_{\beta x} = P_n(\cos \beta x - \sin \beta x)\ e^{-\beta x}$;

$P_n$ = load on an axle at distance x from the mid-point between diaphragms;

$$\beta x = \left(\frac{Kx^4}{EI_{yy}}\right)^{0.25};$$

*K* = defined in a).

**9.3.3** *Distribution of Distortional Warping Stress*

Stress due to restraint of distortional warping should be assumed to be distributed over the cross-section as shown in **Fig. 9.2**. In **Fig. 9.2**, *Bc* is the width of the flange projection beyond the web, or, where there are two or more boxes in one structure, half the clear width of flange between boxes.

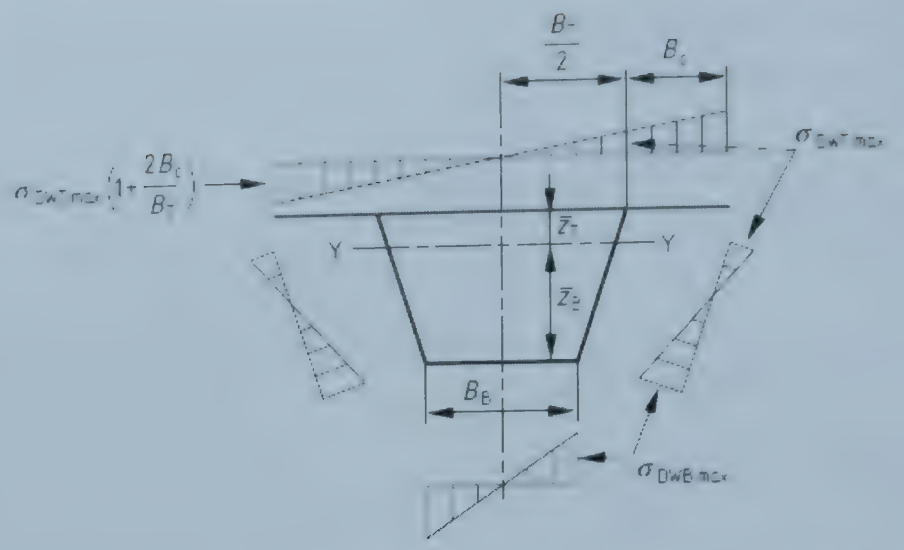



**Fig. 9.2 Longitudinal Stresses due to Distortional Warping**

**9.3.4** *Effective Diaphragm and Cross Frames*

**9.3.4.1** *General*

To be effective for the purposes of this clause, a cross frame or a diaphragm should be such as to satisfy the conditions given in **Clause 9.3.4.2** and **Clause 9.3.4.3**, where the load effects mentioned should be considered as acting in combination with all other simultaneously acting loading effects.

**9.3.4.2** *Strength*

a) A plate diaphragm should be capable of resisting a shear stress $\tau_D$ given by:

$\tau_D = T/2BDt_p$

where:

$t_p$ is the thickness of the diaphragm plate;

$B$ is the average of the widths at the top and bottom flanges;

$D$ is as defined in **Clause 9.2.3**;

$T$ is the torque due to loads applied at the diaphragm and between the adjacent diaphragms or cross frames on either side. Any torque applied between the diaphragm and an adjacent diaphragm or cross frame may be apportioned by simple static analysis.

b) A cross frame consisting of a pair of cross braces connecting both pairs of opposite corners of the box, in which both braces are considered to be simultaneously effective, should be capable of carrying a force $F_B$ in each brace, given by:

$F_B = \frac{T\, L_P B_B}{4\, B\, D\, B_T}$;

where:

$T$, $B$, $D$ are as defined in a);

$B_B$, $B_T$ are as defined in **Clause 9.2.3**;

$L_p$ is as defined in **Clause 9.3.4.3**.

c) A vee-braced cross frame with the vee centred in the top or bottom flange, in which both braces are considered to be simultaneously effective, should be capable of carrying a force $F_B$ in each brace, as given by:

$F_B = \frac{T\, L_b B_B}{2\, B\, D\, B_T}$;

where:

$T$, $B$, $D$ are as defined in a);

$L_b$ is as defined in **Clause 9.3.4.3**;

$B_B$, $B_T$ are as defined in **Clause 9.2.3**.

**9.3.4.3** *Stiffness*

A cross frame or a diaphragm should have a dimensionless stiffness, $S$ not less than the value obtained from **Table 9.1**.

where:

$S = \frac{G t_d L_p^2 \delta_b^2 K}{2 A_p L_D}$ is for plated diaphragm;

$S = \frac{E A_b \delta_b^2 K}{L_D L_p}$ is for a cross braced cross frame;

$S = \frac{E A_b L_p^2 \delta_b^2 K}{4 L_{Dp} L_b^3}$ is for a vee braced cross frame, irrespective of whether the centre of the V is at the top or bottom flange;

$S = \frac{K_R}{K L_D}$ is for an unbraced ring cross frame with constant section framing members; alternatively,

$S = \frac{P_P \delta_b^2 K}{\Delta_p L_D}$ is for any type of cross frame including a ring cross frame;

$\Delta_p$ is the change in length of the diagonal $L_p$ calculated to occur under the system of diagonal forces $P_P$ as shown in **Fig. 9.3**. This method of deriving stiffness may be used for any type of frame including those given above.

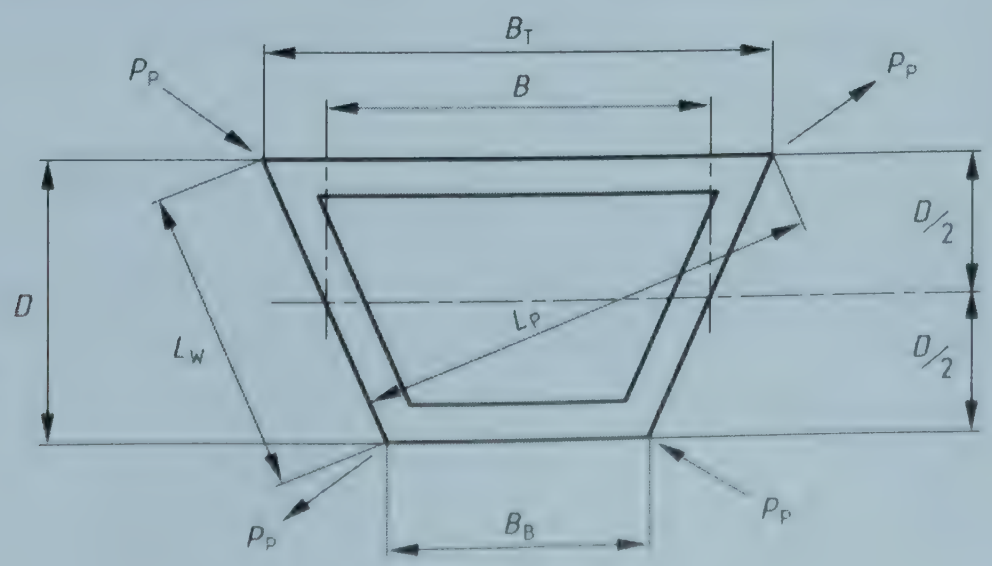



**Fig. 9.3 System of Diagonal Forces**

$L_p = \sqrt{(D2 + B2)}$, the length of the diagonal;

$L_b$ is the length of the brace;

$A_p$ is the surface area of the plated diaphragm, given by $BD$;

$A_b$ is the area of the cross-section of the brace;

$\delta_b$ is the unit length flexibility given by $\frac{4\,B\,D}{K\,B_B L_p}$;

$B$ is the average width of the box girder given by $(B_T + B_B)/2$;

$L_D$ and $K$ are as defined in **Clause 9.3.2**;

$K_R$ is the value of $K$ derived by taking $D_{YB}$, $D_{YC}$ and $D_{YT}$ as the flexural rigidities of the effective framing members attached to the top and bottom flanges and webs respectively;

$B_T$, $B_B$ and $D$ are as defined in **Clause 9.2.3**.

**Table 9.1 Diaphragm Stiffness, *S***

| Single Torque | | | Uniformly distributed torque | |
|---|---|---|---|---|
| $\beta L_D$ | For $\sigma_{DW}$ | For $\sigma_{DB}$ | For $\sigma_{DW}$ | For $\sigma_{DB}$ |
| 3,0 | 0 | 0 | 0 | 5 |
| 2,0 | 0 | 5 | 0 | 30 |
| 1,5 | 2 | 10 | 10 | 200 |
| 1,0 | 20 | 50 | 100 | 500 |
| 0,8 | 50 | 100 | 200 | 1000 |
| 0,5 | 500 | 1000 | 200 | 10000 |
| 0,3 | 2000 | 10000 | 200 | 20000 |

## 9.4 Transverse Distortional Bending Stresses

### 9.4.1 *General*

When loads are applied to a box girder other than at a cross frame or a diaphragm, the resulting transverse bending stresses in the walls of a box should be calculated on the basis of linear elastic theory. The simplified procedure given in **Clause 9.3.2** may be used provided that the cross frames or the diaphragms are in accordance with **Clause 9.3.4**.

### 9.4.2 *Corner Stresses*

The transverse bending stress $\sigma_{DB}$ may be calculated as follows.

a) At a junction between a transverse stiffener on a web and a transverse stiffener or cross beam on a flange, under a uniformly distributed torque, $T_{UD}$:

$\sigma_{DB} = \frac{T_{UD} F_D}{B_T W}$ when $\beta L_D > 2.65$;

$\sigma_{DB} = \frac{T_{UD} F_D}{2\,B_T W}\left(\frac{\beta L_D}{2,2}\right)^{3,7}$ when $\beta L_D \leq 2.65$;

where,

$W$ is the elastic section modulus, per unit of span length, of the flange or web inclusive of transverse stiffeners or cross beams;

$\sigma_{DB}$ is the maximum distortional bending stress in the part to which $W$ refers;

$F_D$ is given by $\frac{B_T}{2}\left(\frac{B_B}{B_B+B_T} - V_D\right)$ at a top flange junction; or $B_B V_D/2$ at a bottom flange junction.

$$V_D = \frac{\left\{\frac{D_{YT}}{D_{YC}}\frac{d}{B_T}\left(2+\frac{B_B}{B_T}\right)+1\right\}}{\left(\frac{B_T}{B_B}+1\right)\left(1+2\left[\frac{D_{YT}}{D_{YC}}\frac{d}{B_T}\left\{1+\frac{B_B}{B_T}+\left(\frac{B_B}{B_T}\right)^2\right\}\right]+\frac{D_{YT}}{D_{YB}}\left(\frac{B_B}{B_T}\right)^3\right)}$$

b) At a junction between a transverse stiffener on a web and a transverse stiffener or cross beam on a flange, due to a concentrated torque $T$ where an axle or concentrated load is applied mid-way between diaphragms:
$\sigma_{DB} = \frac{TF_D}{2B_T L_D W}\beta L_D$ when $\beta L_D > 2$;

$\sigma_{DB} = \frac{TF_D}{15{,}5B_T L_D W}(\beta L_D)^{3.9}$ when $\beta L_D \leq 2$;

where all symbols are as defined in a).

c) Under UDL and concentrated loading, the effects of the uniformly distributed and concentrated loads should be separately calculated as described in a) and b), and the sum of the resulting stresses used.

d) Under a series of concentrated torques due to axle loads:
$\sum\sigma_{DB} = \sigma_{DB1}\sum(P_{\beta x})$;

where,

$\sigma_{DB1}$ = value of $\sigma_{DB}$ obtained from b) for a unit axle load;

$P_{\beta x} = P_n(\cos \beta x + \sin \beta x)\, e^{-\beta x}$

$P_n$, $\beta x$ are as defined in **Clause 9.3.2(d)**.

## 9.5 Distribution of Distortional Bending Stress

Transverse distortional bending moments should be assumed to be distributed over the cross-section, as shown in **Fig. 9.4**, and the resulting stresses calculated using the appropriate value of $W$ at each section.

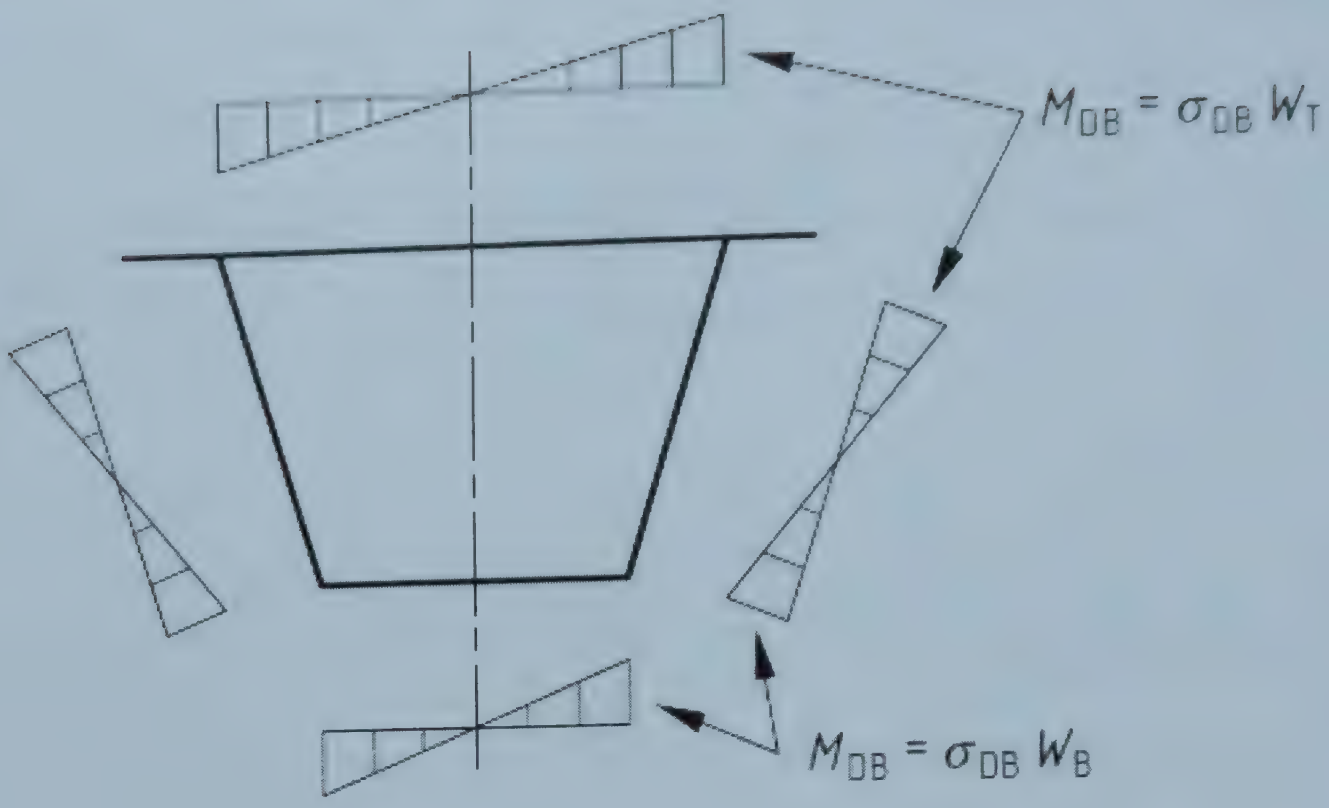



**Fig. 9.4 Transverse Distortional Moments**

# 10. SHEAR CONNECTORS

The design and detailing of Shear Connectors shall be as per **Clause 606** of IRC:22.

# 11. DIAPHRAGMS

## 11.1 General

Diaphragms in general should be designed using finite element analysis, for which specialist literature may be followed. The design methods of **Clause 11.5 & 11.6** use strength provisions that are compatible only with the assumed methods of stress derivation contained therein. Stresses derived by finite element analyses should not be substituted directly for these derived stresses.

a) Diaphragms should be provided at supports of box girders to transfer applied loads to the bearings. Subject to the limitations and provisions of **Clause 11.2**, unstiffened and stiffened diaphragms should be designed in accordance with **Clause 11.5 & 11.6**, respectively, on the basis of the loadings and effective sections given in **Clause 11.3 & 11.4**, respectively.
b) The diaphragm/web junctions should meet the provisions of **Clause 11.7**. Deck cross beams and/or cantilevers supporting the deck and located in the plane of a diaphragm should meet the provisions of **Clause 11.11**.
c) The geometric notation used is shown in **Fig. 11.1**.

## 11.2 Limitations

### 11.2.1 *Box Girders*

Box girders should be of nominally rectangular cross-section or of nominally trapezoidal cross-section with webs in single planes inclined at less than 20° from the vertical, and when unstiffened, should be nominally symmetrical about a vertical axis (i.e., ignoring cross fall or super elevation).

Box girders should be of a single cell form with or without interconnecting cross members and cantilevers.

### 11.2.2 *Diaphragms and Bearings*

The plane of the diaphragm should be within ±5° to the normal to the axis of the girder in elevation, within ±10° in plan, and within ±5° of a vertical plane. The diaphragm should be in a single plane, except as permitted in **Clause 11.2.4** for starter plates.

Each diaphragm should be supported on either a single bearing or twin bearings under each box. Bearings under unstiffened diaphragms should be symmetrically placed about the vertical axis of the diaphragm.

The contact width j of a stiffened diaphragm above a bearing, as defined in **Fig. 11.1**, should not exceed half the depth of the diaphragm with a single bearing nor one-quarter of the depth of the diaphragm with twin bearings. A bearing below a stiffened diaphragm should not extend across the width of the diaphragm beyond the line of attachment of a bearing stiffener by more than $15t_p\varepsilon_p$, where:

$t_p$ is the thickness of the diaphragm plate;

$\varepsilon_p = \sqrt{(250/f_{yp})}$

$f_{yp}$ is the nominal yield strength of the diaphragm plate.

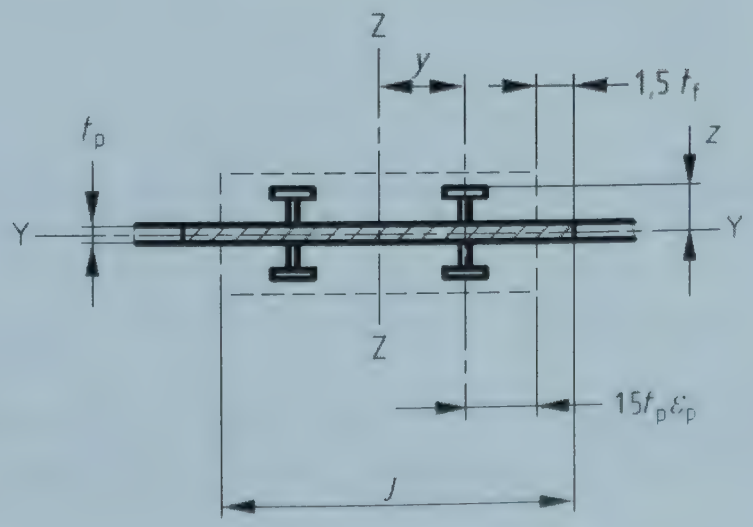



**Fig. 11.1 Geometric Notation for Diaphragms**

### 11.2.3 *Cross Beams and Cantilevers*

Where the deck projects beyond the box web and is supported on cross beams and/or cantilevers which are in the plane of a diaphragm, the flanges of such members should provide

a continuous load path through each box web and across the diaphragm for the forces they are required to carry. These members should be assumed to be supported by the diaphragm/box web junctions (see **Clause 11.7** and **11.8**).

**11.2.4** *Starter Plates*

Where starter plates are to be used to connect a diaphragm to the box walls, they should either be:

a) positioned in the plane of the diaphragm and be butt-welded or connected by double cover plates to the diaphragm; or

b) lap jointed to the diaphragm, provided that a suitable system of stiffening is designed to withstand, in addition to any other load effects, all the moments resulting from the eccentricity of connection.

**11.2.5** *Stiffeners to Diaphragms*

All stiffeners to plate diaphragms should be in accordance with the following:

a) Unless a more advanced method of analysis is carried out in order to prevent torsional buckling of stiffeners with open cross-sections, the following criterion should be satisfied:

$$\frac{I_T}{I_P} \geq 5.3 \frac{f_y}{E}$$

where:

$I_p$ is the polar second moment of area of the stiffener alone around the edge fixed to the plate;

$I_T$ is the St. Venant Torsional Constant for the stiffener alone.

b) Where warping stiffness is considered, stiffeners should either fulfil (a) or the criterion

$\sigma_{cr} \geq \theta f_y$

where:

$\sigma_{cr}$ is the elastic critical stress for torsional buckling not considering rotational restraint from the plate;

$\theta$ is a parameter to ensure class 3 behaviour. The value $\theta = 6$ is recommended.

Bearing stiffeners should be symmetrically placed about the diaphragm plate, unless a special analysis is made of the effects of any eccentricity with respect to that plate.

**11.2.6** *Plating in Diaphragms*

The thickness of plating in an unstiffened diaphragm should be uniform throughout.

**11.2.7** *Openings in Unstiffened Diaphragms*

Openings in unstiffened diaphragms should be in accordance with the following:

a) Only one circular opening may be provided on each side of the vertical centreline of the diaphragm within the upper-third of the height of the diaphragm.

b) The diameter of any such opening should not exceed the least of:

•$6t_p$;

•D/20;

• B/20.

where:

$t_p$ is the diaphragm plate thickness;

D is the depth of the diaphragm;

B is the width of the diaphragm taken as the average of the widths at the top and bottom flange levels for boxes with sloping webs.

c) Cut-outs for longitudinal stiffeners on the box walls should have the stiffeners connected to the diaphragm plate by one of the following methods:

- welding, along at least one-third of the perimeter of the cut-out;
- cleating to the longitudinal stiffener with at least two bolts or rivets per side of the connection, or by full perimeter welding of the cleat.

In addition, the length of the free edge of any cut-out should not exceed $10t_p\varepsilon_p$ when any part of this free edge is within a distance $12t_p\varepsilon_p$ from any part of a bearing plate,

where:

$t_p$ is the diaphragm plate thickness;

$$\varepsilon_p = \sqrt{(250/f_{yp})}$$

$f_{yp}$ is the nominal yield strength of the diaphragm plate.

**11.2.8** *Openings in Stiffened Diaphragms*

**11.2.8.1** *Position of Openings*

With the exception of openings permitted in **Clause 11.2.8.4**, openings should not be positioned within the areas shown shaded in **Fig. 11.2(a)** and **Fig. 11.2(b)**.

**11.2.8.2** *Unstiffened Openings*

Unstiffened openings should be circular and of diameter not exceeding the least of:

•$6t_p$;
•a/20;
•b/20.

except when:

$$\sigma_e \leq \frac{f_{yp}}{2\gamma_{M0}}$$

for which the limiting diameter is twice the limits given in **Clause 11.2.8.3(a), (b)** and **(c);**

where:

a and b are the panel dimensions;

$$\sigma_e = \sqrt{\sigma_{p1}^2 + \sigma_{p2}^2 - \sigma_{p1}\sigma_{p2} + 3\tau_p^2}$$

$\sigma_{p1,}\sigma_{p2}$&$\tau_p$ are the stresses in the diaphragm plate derived in accordance with **Clause 11.6**;

$f_{yp}$ is the nominal yield strength of the diaphragm plate.

Not more than one such opening should be positioned in a single plate panel.

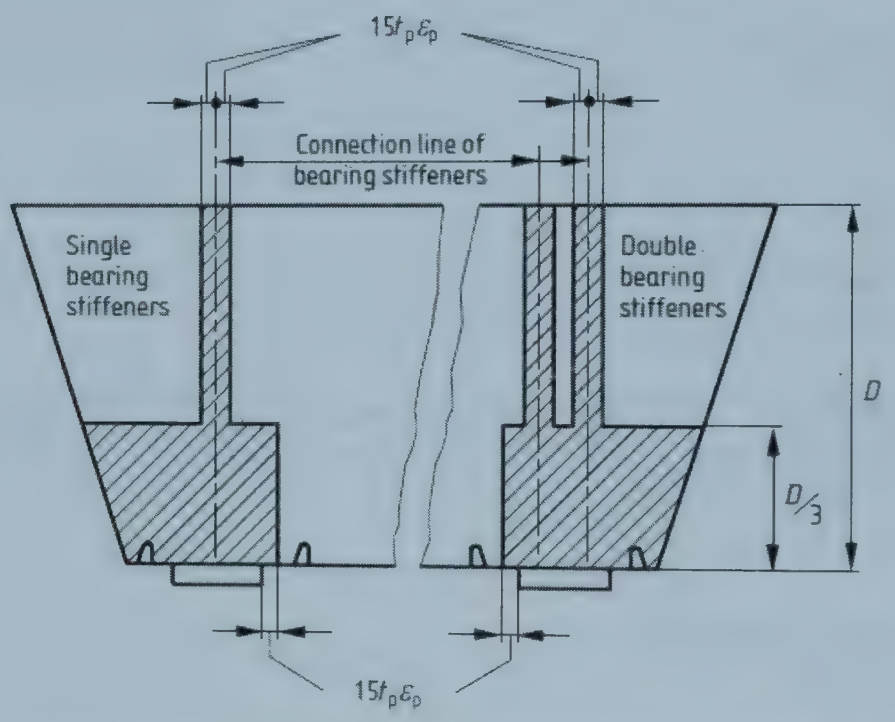
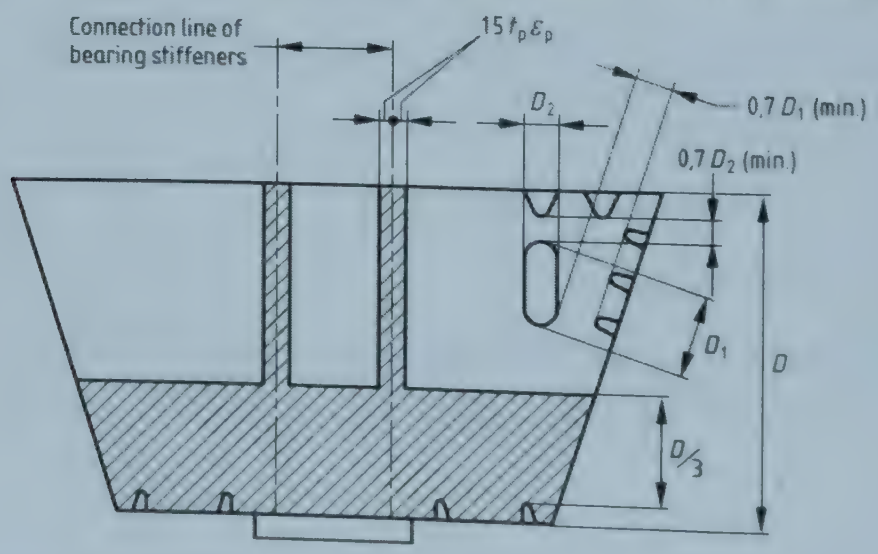



**Fig. 11.2(a) Openings in Stiffened Diaphragms – Twin Bearings**




*NOTE 1 Openings are not permitted in shaded areas.*

*NOTE 2 Dimensions are taken from top of cut-outs where present.*

**Fig. 11.2(b) Openings in Stiffened Diaphragms – Single Central Bearing and Positioning of Bearings**

**11.2.8.3** *Stiffened Openings*

Stiffened openings should:

a) be framed on all sides by stiffeners;

b) have circular corners of radius at least one-quarter of the least dimension of the hole, with no re-entrant corners;

c) be positioned such that the distance of any edge from an adjacent wall of the box is at least 0.7 times the maximum dimension of the hole parallel to the wall, plus the distance from the wall to the tips of any cut-outs in the diaphragm for longitudinal stiffeners (Refer **Fig. 11.2(b)**, unless the adjacent plate is designed for secondary in-plane stresses.

**11.2.8.4** *Cut-outs for Longitudinal Stiffeners*

Cut-outs for longitudinal stiffeners should be in accordance with **Clause 11.2.7(c).**

## 11.3 Loading on Diaphragms

**11.3.1** *Derivation*

The load effects in diaphragms and associated parts of box girders should be derived from global analysis undertaken in accordance with the following:

a) Effects of deformed geometry of the structure:

a.1) The internal forces and moments may generally be determined using either:

- first-order analysis, using the initial geometry of the structure or
- second-order analysis, taking into account the influence of the deformation of the structure.

a.2) The effects of the deformed geometry (second-order effects) should be considered if they increase the action effects significantly or modify significantly the structural behaviour.

a.3) First order analysis may be used for the structure, if the increase of the relevant internal forces or moments or any other change of structural behaviour caused by deformations can be neglected. This condition may be assumed to be fulfilled, if the following criterion is satisfied:

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10, \text{ for elastic analysis}$$

where,

$\alpha_{cr}$ is the factor by which the design loading would have to be increased to cause elastic instability in a global mode

$F_{Ed}$ is the design loading on the structure

$F_{cr}$ is the elastic critical buckling load for global instability mode based on initial elastic stiffness

a.4) The bridges and components may be checked with first order theory if the following criterion is satisfied for each section:

$\alpha_{cr} \geq 10$

a.5) The effects of shear lag and of local buckling on the stiffness should be taken into account if this significantly influences the global analysis.

a.6) The effects on the global analysis of the slip in bolt holes and similar deformations of connection devices like studs and anchor bolts on action effects should be taken into account, where relevant and significant.

b) Structural stability of frames:

b.1) If according to (a) the influence of the deformation of the structure has to be taken into account, (b.2) to (b.5) should be applied to consider these effects and to verify the structural stability.

b.2) The verification of the stability of frames or their parts should be carried out considering imperfections and second order effects.

b.3) According to the type of frame and the global analysis, second order effects and imperfections may be accounted for by one of the following methods:

b.3.1) both totally by the global analysis,

b.3.2) partially by the global analysis and partially through individual stability checks of members according to **Clause No. 507, 509 & 510** of IRC:24,

b.3.3) for basic cases by individual stability checks of equivalent members according to **Clause No. 507, 509 & 510** of IRC:24 using appropriate buckling lengths according to the global buckling mode of the structure.

b.4) Second order effects may be calculated by using an analysis appropriate to the structure (including step-by-step or other iterative procedures). For frames where the first sway buckling mode is predominant, first order elastic analysis should be carried out with subsequent amplification of relevant action effects (e.g. bending moments) by appropriate factors.

b.5) Where the behaviour of a bridge or its components is governed by the first buckling mode (single degree of freedom) the second order effects $M_{II}$ may be calculated by applying a factor to the bending moments $M_I$ as follows:

$$M_{II} = M_I \frac{1}{1 - \frac{1}{\alpha_{cr}}}$$

b.6) In accordance with (b.3) the stability of individual members should be checked according to the following:

b.6.1) If second order effects in individual members and relevant member imperfections are totally accounted for in the global analysis of the structure, no individual stability check for the members according to **Clause 507, 509 & 510** of IRC:24 is necessary.

b.6.2) If second order effects in individual members or certain individual member imperfections (e.g. member imperfections for flexural and/or lateral torsional buckling) are not totally accounted for in the global analysis, the individual stability of members should be checked according to the relevant criteria in **Clause 507, 509 & 510** of IRC:24 for the effects not included in the global analysis. This verification should take account of end moments and forces from the global analysis of the structure, including global second order effects and global imperfections when relevant and may be based on a buckling length equal to the system length

b.7) Where the stability of a frame is assessed by a check with the equivalent column method according to **Clause 507, 509 & 510** of IRC:24 the buckling length values should be based on a global buckling mode of the frame accounting for the stiffness behaviour of members and joints, the presence of plastic hinges and the distribution of compressive forces under the design loads. In this case internal forces to be used in resistance checks are calculated according to first order theory without considering imperfections.

**11.3.2** *Effects to be Considered*

Diaphragms should be designed to resist, with due account being taken of any lack of symmetry in the cross-section or in the bearing arrangement, the combined effects of the following:

a) All externally applied loads and the associated bearing reactions.

b) Changes in bearing reactions and web shears due to.

1) creep, shrinkage and differential temperature;

2) settlement and other movement of supports.

Transverse effects due to b) (1) may be neglected.

c) Errors in installation of bearings, comprising:

1) bearing misalignment in plan;

2) errors in level of a single bearing, or in the mean levels of more than one bearing at any support;

3) bearing inclination;

4) departures from common planarity of twin or multiple bearings.

d) Changes in longitudinal slope of box flanges at the diaphragm.

e) Errors of longitudinal camber in continuous construction.

Allowance for this may be made by assuming, at the bearings, a vertical displacement of a support relative to two adjacent supports of 1/5000 times the sum of the adjacent spans.

f) Out-of-plane moments due to any or all of the following, as appropriate:

1) longitudinal movements of the bridge;

2) changes in slope of the bridge;

3) eccentricity due to bearing misalignment along the span or due to the shape of the bearing; the combined eccentricity for these may be taken as:

- half the width of the flat bearing surface plus 10 mm for flat topped rocker bearing in contact with flat bearing surface; or
- 3 mm for radiused upper bearing resting on flat or radiused lower part; or
- 10 mm for flat upper bearing resting on radiused lower part.

4) interconnection between deck and diaphragm stiffeners;

5) any intended eccentricity of the centroidal axes of the effective section of the bearing stiffeners with respect to the diaphragm plate.

## 11.4 Effective Sections

### 11.4.1 *General*

For determining the stresses in a diaphragm, the effective elastic section modulus and effective area of a vertical cross-section, and the effective vertical and horizontal shear areas, should be derived in accordance with **Clause 11.4.2** and **11.4.3**. For determining the stresses in stiffeners, their effective sections should be derived in accordance with **Clause 11.4.4** or **11.4.5**, as appropriate.

In **Clause 11.4.2** and **11.4.3** the references to transverse tension and compression apply to directions normal to the longitudinal axis of the girder.

### 11.4.2 *Vertical Sections*

#### 11.4.2.1 *General*

The determination of the effective area $A_e$ and the effective section modulus $W_e$ of a vertical cross-section of a diaphragm, should be based on effective areas of box flanges and diaphragm plate as given in **Clause 11.4.2.2 to 11.4.2.5**.

#### 11.4.2.2 *Effective Flange Width*

In calculating an effective area of a box flange, an effective width should be determined separately for each side of the diaphragm and should not exceed any of the following:

a) one-quarter of the distance of the section under consideration from the nearest web/flange junction;

b) half the distance to an adjacent diaphragm or cross beam for any flange in transverse tension, or for a composite flange in transverse compression;

c) outside an end diaphragm, the actual width of plate provided;

d) $15t_f\varepsilon_f$ for a non-composite flange in transverse compression. This limit may be increased to one-quarter of the distance to an adjacent diaphragm or cross beam provided that the transverse compressive stress (using the increased width) does not exceed the lesser of:

- one-quarter of the longitudinal compressive strength of the flange;
- $0.5\left(\frac{t_f}{b}\right)^2 E$

where:

$t_f$ is the thickness of the flange plate;

$\varepsilon_f = \sqrt{(250/f_{yf})}$

$f_{yf}$ is the nominal yield strength of the flange plate;

$b$ is the spacing of the longitudinal flange stiffeners or the distance between box webs for an unstiffened flange.

**11.4.2.3** *Effective Flange Area*

The effective area of a box flange should be determined as follows.

The effective area of steel plate on each side of the diaphragm should be taken as $K_c t_f b_e$;

where:

$t_f$ is the flange thickness;

$b_e$ is the effective width on the appropriate side of the diaphragm derived from **Clause 11.4.2.2**;

$K_c$ is a coefficient taken as 1.0, except in the case of a non-composite flange in transverse compression with an effective width greater than $15t_f\varepsilon_f$. The value of $K_c$ should be obtained from **Fig. 11.3** with the dimension $a$ taken as the spacing of longitudinal flange stiffeners and dimension $b$ taken as the distance from the diaphragm to an adjacent cross beam or diaphragm. In using **Fig. 11.3**, the restrained curve should be used for diaphragms at internal supports of continuous beams and the unrestrained curve for diaphragms at end supports;

Any transverse flange stiffeners within the effective width should be ignored.

In composite construction, the effective flange area may include the area of steel reinforcement within the total effective width, and, if subjected to transverse compression, may also include the transformed area of concrete within the total effective width.

**11.4.2.4** *Diaphragm Plate*

Holes within the vertical section of a diaphragm should be deducted. When a stiffened opening is provided, diaphragm plating extending within the framing stiffeners by more than $10t_p\varepsilon_p$ should be ignored,

where:
$\varepsilon_p = \sqrt{(250/f_{yp})}$;

**11.4.2.5** *Inclined Webs*

In the case of box girders with inclined webs, no part of the webs should be included in the vertical section of a diaphragm.

**11.4.3** *Shear Area*

The effective vertical and horizontal shear areas $A_{ve}$ and $A_{he}$ should be taken as the net areas of a vertical and horizontal cross-section, respectively, of diaphragm plating only.

**11.4.4** *Diaphragm Stiffeners*

a) The effective section of a stiffener on a diaphragm should be taken to comprise the stiffener with widths of diaphragm plate on each side of the stiffener where available, not exceeding the lesser of:
   • half the distance from the stiffener to an adjacent stiffener or to the wall of the box;
   • $15\varepsilon_p$ times the thickness of the diaphragm plate.

b) Additionally, for a bearing stiffener, the effective width of plate assumed on the side towards the web should not exceed half the distance from the stiffener to the web/bottom flange junction.

c) The sectional area of discontinuous diaphragm stiffeners should be ignored.

**11.4.5** *Diaphragm/Web Junction*

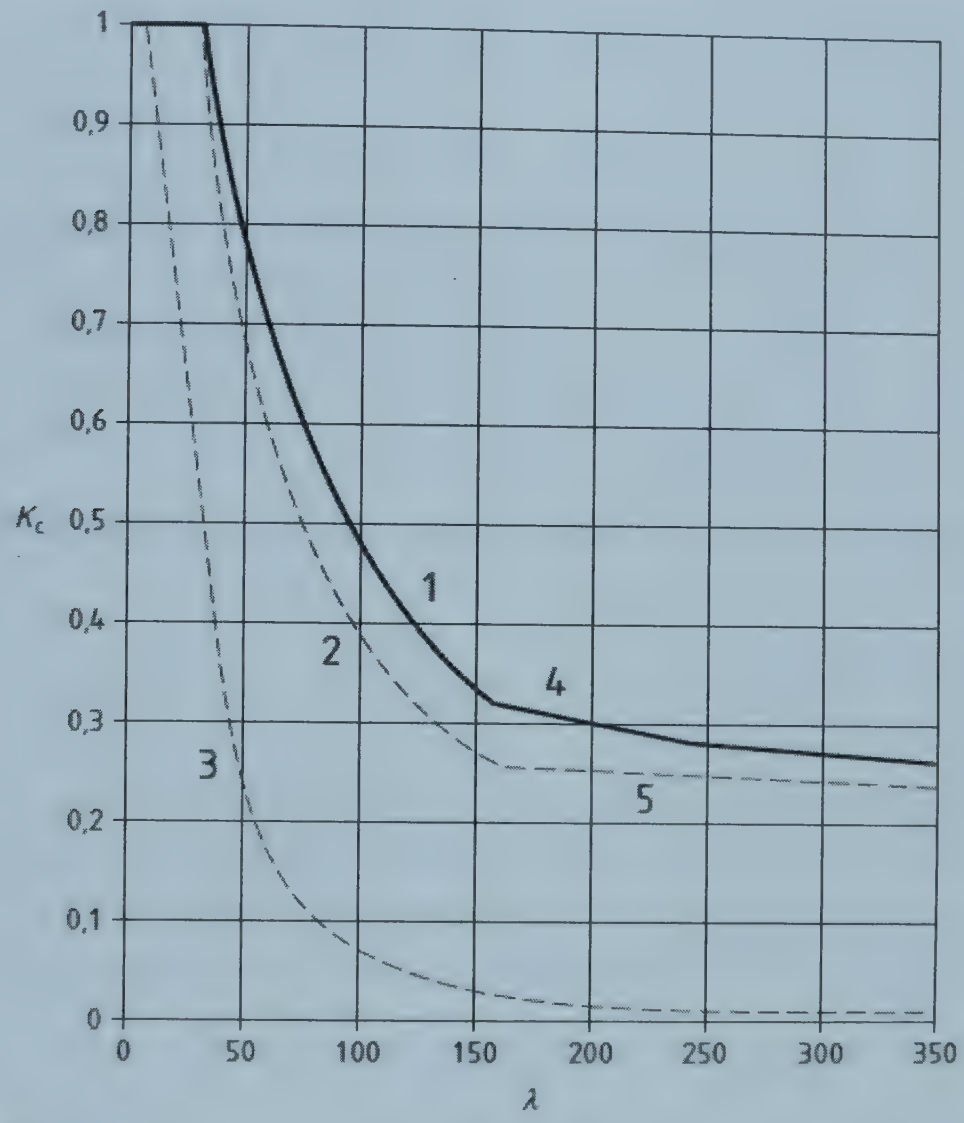
The effective section of this part should be taken to comprise a) below, plus b), c), or d) as appropriate:

a) a width of web plating each side of the diaphragm (where available) of up to 16 times the web thickness;
b) where there is a load bearing stiffener on the diaphragm, within a distance of $25t_p$ from the web: a width of diaphragm plate equal to half the distance between the web and the load bearing stiffener;
c) where there is a stiffener, other than a load bearing stiffener, on the diaphragm parallel to, and within a distance of $25t_p$ from the web: the area of this stiffener together with a width of the diaphragm plate equal to $25t_p$;

d) when there is no stiffener on the diaphragm parallel to, and within a distance of $25t_p$ from, the web: a width of diaphragm plate equal to $15t_p\varepsilon_p$;
where:

$t_p$ is the thickness of the diaphragm plate;

$\varepsilon_p$ is as defined in **Clause 11.4.2.4**.

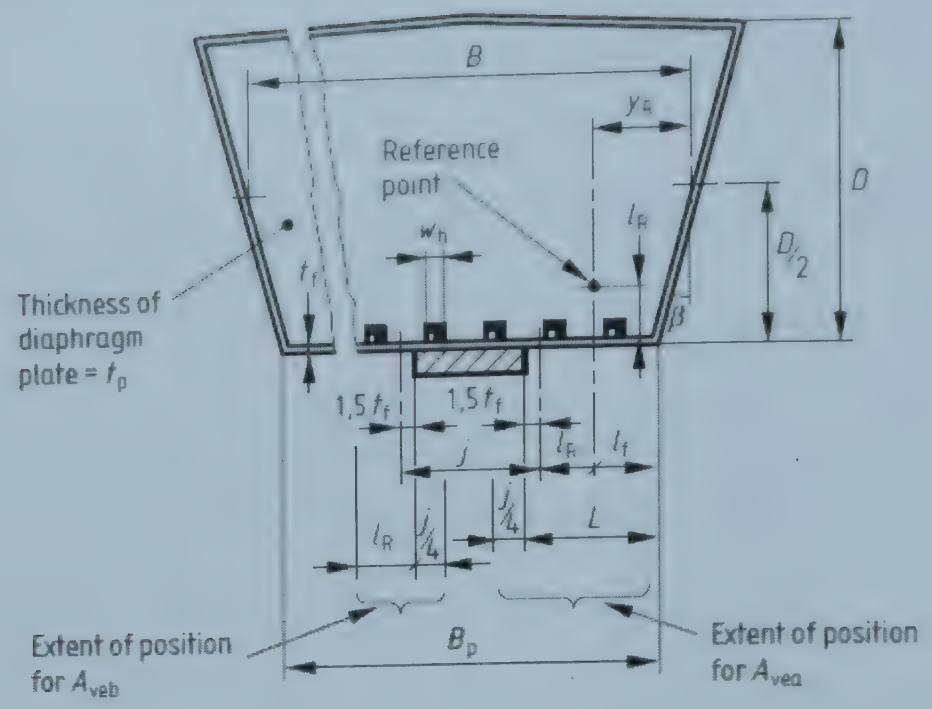



**Key**
1 Curve 1
2 Curve 2
3 Curve 3
4 Restrained
5 Unrestrained

**Fig. 11.3 Coefficient $K_c$ for Plate Panels under Direct Compression** [6]

The value of $K_c$ to be used is the higher of the values obtained using either:

a) curve 1 or 2 as relevant, with $\lambda = b/(t_f\varepsilon_f)$;
b) curve 3, with $\lambda = a/(t_f\varepsilon_f)$;
where:
a is the panel dimension in the direction of stress considered;
b is the panel dimension normal to the direction of stress;
a will always give the higher value for $K_c$ when $a/b \geq 0.5$.
For $a/b < 0.5$, (a) or (b) may give a higher value.




*NOTE* $l_R$ *is taken as the least of:*

$7.5t_p\varepsilon_p$ $\frac{D}{8}$ or $\frac{L}{2}$.

**Fig. 11.4 Reference Point and Notation for Unstiffened Diaphragms**

## 11.5 Unstiffened Diaphragms

### 11.5.1 *General*

Unstiffened diaphragms in accordance with **Clause 11.2.1** to **11.2.4, 11.2.6** and **11.2.7**, should be designed to meet the yield criteria of **Clause 11.5.4** and the buckling criterion of **Clause 11.5.5** using reference stress values of **Clause 11.5.2** and the buckling coefficients of **Clause 11.5.3**. Web/flange junctions should additionally be in accordance with **Clause 11.7.3**.

### 11.5.2 *Reference Values of In-Plane Stresses*

#### 11.5.2.1 *General*

The stresses in an unstiffened diaphragm, resulting from the load effects given in **Clause 11.3**, should be determined at the reference point indicated in **Fig. 11.4**, in accordance with **Clause 11.5.2.2** to **Clause 11.5.2.4**, for each of the appropriate reference stresses required.

#### 11.5.2.2 *Vertical Stresses*

The reference value of the in-plane vertical stress $\sigma_{R1}$ should be taken as follows:

a) for a diaphragm with a single central bearing:

$$\sigma_{R1} = \frac{R_v\,(1 + 4e\,/\,t_p\,)}{(j - \Sigma w_h)\,t_p} + 0.77\left(\frac{T_b\,j}{2\,I_{zp}}\right);$$

b) for a diaphragm with a pair of twin symmetrical bearings:

$$\sigma_{R1} = \frac{R_v\,(1+4e\,/\,t_p\,)}{(j-\Sigma w_h)\,t_p};$$

where:

$R_v$ is the total vertical load transmitted by the diaphragm to one bearing (including the effects of torque on twin bearings);

$T_b$ is the torsional reaction at a single central bearing;

$J$ is the width of contact of the bearing pad plus 1.5 times the thickness of the bottom flange at each end if available (Refer **Fig. 11.1);**

$\Sigma w_h$ is the sum of the widths of any cut-outs for stiffeners within the width J at the level immediately above the flange;

$t_p$ is the thickness of the diaphragm plate;

$I_{zp}$ is the second moment of area of the diaphragm plate of width $j$ excluding cut-outs, about the Y-axis (Refer **Fig. 11.1**);

$e$ is the eccentricity of bearing reaction along the span, which should include the effects of:

- movements of the beam relative to the bearing due to changes in temperature;
- changes in the point or line of contact at the spherical or cylindrical surface of a bearing due to slope of the beam when deflected by load;
- uneven seating which may occur on a flat bearing surface;
- inaccuracy which may occur in positioning of the beam relative to the bearing.

Values of eccentricity to satisfy c) and d) may be taken from **Clause 11.3.2 (f)(3)**.

**11.5.2.3** *Horizontal Stresses*

The reference value of the in-plane horizontal stress $\sigma_{R2}$ should be taken as:

$$\sigma_{R2} = \left[\left(\frac{K_p \Sigma R_v}{2} + \frac{T}{B}\right) y_R + Q f_v \frac{l_f}{2}\right] \frac{1}{W_e} + \frac{\Sigma R_v \tan\beta}{2A_e};$$

where:

$K_p$ is a factor to allow for the effects of boundary shears and should be taken as 2.0 in the absence of any special analysis;

$\Sigma R_v$ is the total vertical force transmitted by the diaphragm to the bearings;

$Qf_v$ is the vertical force transmitted to the diaphragm by the portion of the bottom flange over a width $l_f$ when there is a change of flange slope;

$l_f$ is the horizontal distance from the reference point to the nearest edge of the bottom flange;

$\beta$ is as defined in **Clause 11.2.7**;

$T$ is the torque transmitted to the diaphragm in shear through the box walls and from cross beam and/or cantilever loading;

$y_R$ is the distance parallel to the bottom flange from the reference point to the web mid-point (Refer **Fig. 11.4**);

$W_e$ and $A_e$ are the effective section modulus and the effective area respectively of the diaphragm and flanges at the vertical cross-section through the reference point, derived in accordance with **Clause 11.4.2**;

$\beta$ is the inclination of the box web to the vertical.

**11.5.2.4** *Shear Stresses*

**11.5.2.4.1** *Reference Value of In-Plane Shear Stresses*

Except as required by **Clause 11.5.2.4.2**, the reference value of the in-plane shear stress R should be taken as follows:

$$\tau_R = \left(\frac{\Sigma R_v}{2} + Q_{fv} + \frac{T}{2B}\right)\frac{1}{A_{vea}} + \frac{Q_h}{A_{he}};$$

where:

$R_v$, $Q_{fv}$ and $T$ are as defined in **Clause 11.5.2.3**;

$B$ is as defined in **Clause 11.2.7**;

$Q_h$ is the shear force due to transverse horizontal loads on the bridge transmitted from the top flange to the diaphragm;

$A_{vea}$ is the minimum value of the effective vertical shear area, as given in **Clause 11.4.3**, for any section of diaphragm plating taken between the web and a point $j/4$ inside the outer edge of the bearing **(Refer Fig. 11.4);**

$j$ is as defined in **Clause 11.5.2.2**;

$A_{he}$ is the effective horizontal shear area, as given in **Clause 11.4.3** for the section of diaphragm plating through the reference point.

**11.5.2.4.2** *Diaphragms on Twin Symmetrical Bearings*

In addition to the values derived from **Clause 11.5.2.4.1**, in the case of diaphragms on twin symmetrical bearings where there is a change in slope of the bottom flange, an alternative value $\tau_{Rf}$ should be derived from:

$$\tau_{Rf} = \left(\frac{T}{c} + \frac{Q_{bv}}{2} - \frac{T}{2B}\right)\frac{1}{A_{veb}} + \frac{Q_h}{A_{he}}$$

where:

$T$ is as defined in **Clause 11.5.2.3**;

$Q_{bv}$ is the total vertical force transmitted to the diaphragm by the portion of the bottom flange between the inner edges of the bearings when there is a change in flange slope;

$c$ is the distance between centres of bearings;

$A_{veb}$ is the minimum value of the effective vertical shear area, as given in **Clause 11.4.3**, for any section of diaphragm plating taken within a distance $l_R$ from the inner edge of a bearing (i.e. towards the diaphragm centreline) and a distance $j/4$ inside the same inner edge of the bearing

**(**Refer **Fig. 11.2(a)** and **Fig. 11.2(b).**

$l_R$ is as defined in **Fig. 11.4**.

This value $\tau_{Rf}$ should be adopted if it exceeds the value of $\tau_R$ determined in **Clause 11.5.2.4.1**.

**11.5.3** *Buckling Coefficient*

In checking the adequacy of an unstiffened plate diaphragm, a coefficient $K$ is required which is given by:

$K = K_1K_2K_3K_4$

where:

$K_1 = 3.4 + (2{,}2\ D / B_p)$;

$K_2$ for single central bearings;

$K_2 = 0.4 + (j / 2B_P)$;

$K_2$ for twin bearings;

$K_1 = 0.4 + (c - j/3)/B_p$;

$K_3 = 1.0 - \beta/100$;

$$K_4 = 1.0 - \frac{f\ P_p}{\sum R_v + T\ /\ l_b}\left(\frac{2\ B}{B_p} - 1\right);$$

$D$, $B_p$, $B$ and $\beta$ (in degrees) are as defined in **Fig. 11.4**;

$j$ is as defined in **Clause 11.5.2.2**;

$f$ = 0.55 when $D/B \leq$ 0.7 or 0.86 when $D/B \geq$ 1,5 with intermediate values found by linear interpolation;

$l_b = j/2$ for single central bearings, or c for twin bearings;

$\Sigma R_v$ and $T$ are as defined in **Clause 11.5.2.3**;

$c$ is the distance between centres of bearings;

$$P_p = F_p + \sum\left(\frac{P}{K_5}\right);$$

$F_p$ is the total uniformly distributed load applied to the top of the diaphragm;

$P$ is any local load applied to the top of a diaphragm;

$K_5 = 0{,}4 + (w /(2B - B_p))$;

$w$ is the actual width of the load $P$ plus an allowance for the dispersal through a concrete flange at an angle of 45° to the vertical, and through a steel flange at an angle of 60° to the vertical.

**11.5.4** *Yielding of Diaphragm Plate*

The value of $\sigma_{R1}$ and $\sqrt{(\sigma_{R2}^2 + 3\tau_R^2)}$ should not exceed the lesser of:

$f_{yp} / \gamma_{M0}$; or

$$\frac{f_{yp}}{\gamma_{M0}}\left[1{,}2 - \frac{(\sum R_v + T / l_b)D}{1{,}25KEt_p^3}\right];$$

where:

$\sigma_{R1}$, $\sigma_{R2}$ and $\tau_R$ are the reference values of stress as derived in **Clause 11.5.2.2**, **11.5.2.3** and **11.5.2.4**, respectively;

$\sum R_v$ and $T$ are as defined in **Clause 11.5.2.3**;

$l_b$ and $K$ are as derived in **Clause 11.5.3**;

$D$ is as defined in **Fig. 11.4**;

$t_p$ and $f_{yp}$ are, respectively, the thickness and nominal yield strength of the diaphragm plate

**11.5.5** *Buckling of Diaphragm Plate*

The value of $\Sigma R_v + T/l_b$ should not exceed $0{,}7KEt_p^3 / (D\ \gamma_{M1})$;

where:

$l_b$ and K are as derived in **Clause 11.5.3**;

$t_p$ is the thickness of the diaphragm plate;

$D$ is as defined in **Fig. 11.4**;

$\Sigma R_v$ and $T$ are as defined in **Clause 11.5.2.3**

## 11.6 Stiffened Diaphragms

**11.6.1** *General*

a) Diaphragms in accordance with **Clause 11.2.1** to **11.2.6** and **11.2.8** and stiffened by an orthogonal system of stiffeners, generally as indicated in **Fig. 11.1**, should be designed such that the diaphragm plate meets the yield criterion of **Clause 11.6.4** and the buckling criterion of **Clause 11.6.5**, using the appropriate stresses determined from **Clause 11.6.2**.

b) In addition, all types of stiffeners, as defined in a), b) and c) below, should be designed such that they meet the yield criterion of **Clause 11.6.6** and the buckling criterion of **Clause 11.6.7**, using the appropriate stresses determined from **Clause 11.6.3**.

c) Web/flange junctions should, additionally, be in accordance with **Clause 11.7.3** and **11.7.4**.

d) Stiffening may consist of (Refer **Fig. 11.4**):

- bearing stiffeners, which span from a box flange immediately above a bearing, to the flange at deck level;
- stub stiffeners, which are short vertical stiffeners above bearings;

- intermediate stiffeners, which may be either primary or secondary. Stiffeners spanning between box walls or, if horizontal, between a box web and a bearing stiffener, or between bearing stiffeners should be treated as primary. All other stiffeners should be treated as secondary.

**11.6.2** *Stresses in Diaphragm Plates*

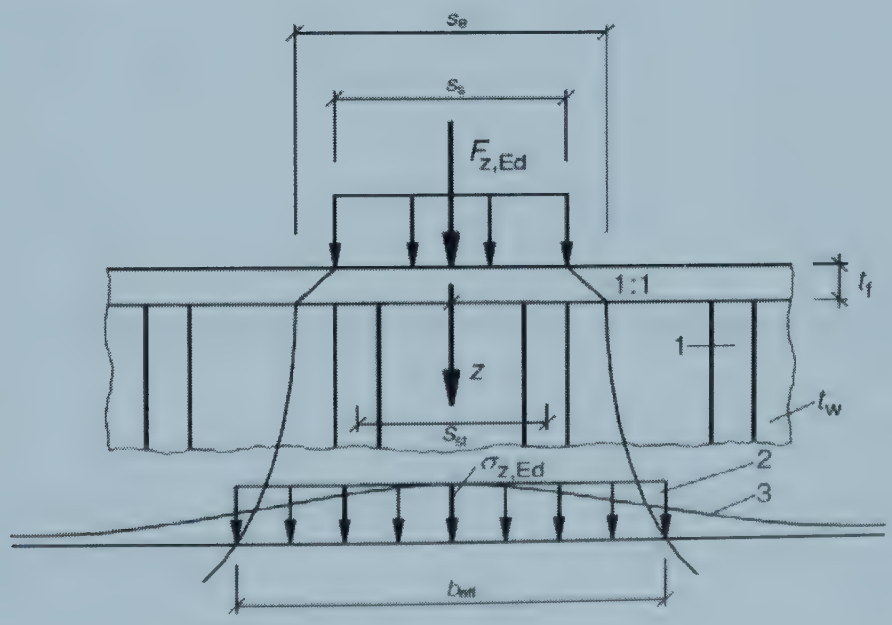
**11.6.2.1** *General*

Relevant stress components should be calculated at the corners of each plate panel, using the appropriate section properties obtained from **Clause 11.4**, in accordance with **Clause 11.6.2.2** to **11.6.2.4**. When secondary bending stresses have been calculated in accordance with **Clause 11.2.8 (c)(3)** they should be added to these components.

**11.6.2.2** *Vertical Stresses*

Vertical stresses $\sigma_{pl}$ may be neglected with the exception of those due to:

a) a change in slope of the main girder flange; and
b) local wheel loads applied above the diaphragm, which should be calculated in accordance with follows:




*1 stiffener*
*2 simplified stress distribution*
*3 actual stress distribution*

The elastic stress distribution in a stiffened or unstiffened plate due to the local introduction of in-plane forces (patch loads), should be determined from:

$\sigma_{z,Ed} = \frac{F_{Ed}}{b_{eff}\,(t_w + a_{st,l})}$;is valid when $s_{st}/s_e \leq 0.5$; otherwise the contribution of stiffeners should be neglected.

with:

$$b_{eff} = s_e \sqrt{1 + \left(\frac{z}{s_e n}\right)^2};$$

$$n = 0{,}636\sqrt{1 + \frac{0{,}878\, a_{st,l}}{t_w}};$$

$s_e = s_s + 2\, t_f$

where,

$a_{st,l}$ is the gross cross-sectional area of the stiffeners smeared over the length $s_e$. This may be taken, conservatively, as the area of the stiffeners divided by the spacing $s_{st}$;

$t_w$ is the web thickness;

z is the distance to flange.

The maximum value within a panel may conservatively be assumed to act over the entire length of the longitudinal edges of the panel.

**11.6.2.3** *Horizontal Stresses*

**11.6.2.3.1** *General*

Horizontal stresses $\sigma_{p2}$ should be calculated under the action of the following Clauses (**Clause 11.6.2.3.2** and **11.6.2.3.3**).

**11.6.2.3.2** *The in-plane primary moment on the diaphragm*

The in-plane moment M should be taken as:

$$M = (K_p Q_v + 2Q_T) y_w + K_p Q_c y_c + \sum_{i=1,n}(P_i\, y_i) - R_v y_b + \left(\frac{Q_{fv}\, l_f}{2}\right);$$

where (as shown in **Fig. 11.5**):

$Q_v$ is the total vertical component of symmetric shear transmitted into the diaphragm from one web;

$Q_T$ is the vertical component of torsional shear transmitted into the diaphragm from one web, given by *T/(2B);*

*B* is as defined in **Clause 11.2.7**;

In calculating the effects in plate panels occurring in a region bounded by an inclined girder web, a girder top flange, and a vertical line passing through the bottom flange/web junction, an appropriate portion of $Q_v$ and $Q_T$ should be taken on the assumption that they are uniformly distributed over the depth of the girder web.

$y_w$ is the horizontal distance, from the section under consideration to the midpoint of the web;

$Q_C$ is the vertical component of any cross beam or cantilever shear;

$y_c$ is the horizontal distance from the section under consideration to the root of the cross beam or cantilever;

$P_i$ is a locally applied deck load between the section under consideration and the web;

$y_i$ is the horizontal distance from the section under consideration to the locally applied deck load $P_i$;

$R_v$ is the total vertical load transmitted to one bearing by the diaphragm;

$y_b$ is the distance from the section under consideration to the inner edge of the nearest bearing plus $j/4$ for sections between twin bearings, or is zero for all other sections, and for diaphragms with a single bearing;

$K_p$, $Q_{fv}$ and $l_f$ are as defined in **Clause 11.5.2.3**;

$j$ is as defined in **Clause 11.5.2.2**.

The horizontal bending stress $\sigma_{2b}$ should be taken as:

$\sigma_{2b} = M/W_e$;

where:

$W_e$ is the effective section modulus of a vertical cross-section of the diaphragm and flanges, at the point under consideration, derived in accordance with **Clause 11.4.2**.

**11.6.2.3.3** *The horizontal component of girder shear when the webs are inclined*

a) The horizontal stress $\sigma_{2q}$ from this component should be taken as:

$\sigma_{2q} = Q_v \tan \beta / A_e$;

where:

$Q_v$ is as defined in **Clause 11.6.2.3.2**;

$A_e$ is the effective area of a vertical cross-section of the diaphragm and flanges, at the point under consideration, derived in accordance with **Clause 11.4.2**;

$\beta$ is the inclination of the box web to the vertical.

b) The total horizontal stress $\sigma_{p2}$ at the point under consideration should be taken as:

$\sigma_{p2} = \sigma_{2b} + \sigma_{2q}$

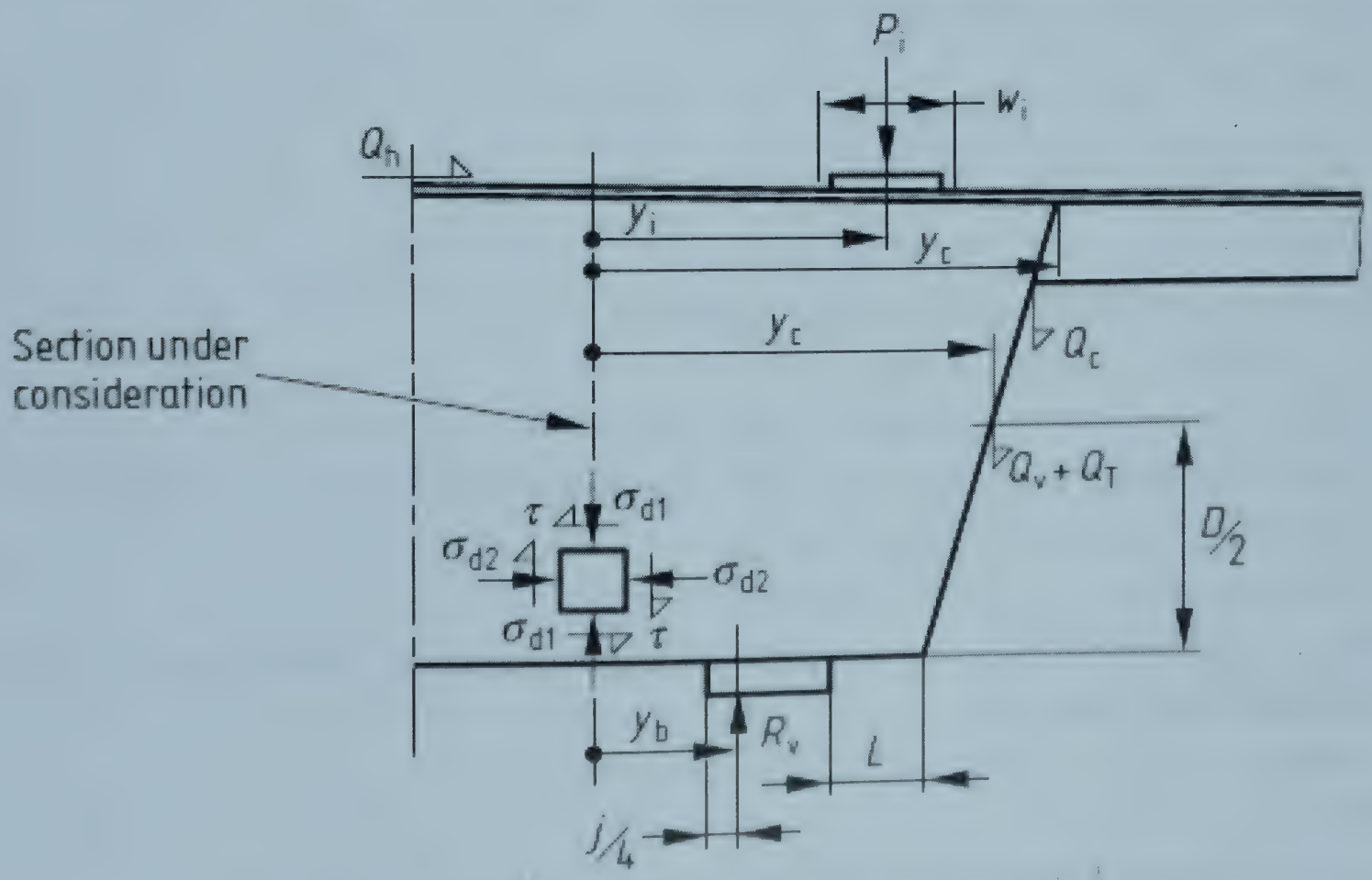



**Fig. 11.5 Load Effects and Notation for Stiffened Diaphragms**

**11.6.2.4** *Shear Stresses*

**11.6.2.4.1** Shear stresses should be calculated under the action of the shear flow $q$ at the section of the diaphragm under consideration. This shear flow $q$ should be taken as constant over the net depth or width of the diaphragm, and as follows:

a) In sections between a box web and an outer bearing stiffener:

$$q = (Q_v + Q_T + Q_{fv} + Q_c + \sum P_i)/D_e + Q_h/B_e$$

b) In sections between inner bearing stiffeners where there are twin bearings:

$$q = (Q_v/4 + Q_{bv}/2 + T/c - Q_T)/D_e + Q_h/B_e$$

c) In sections between pairs of bearing stiffeners above one of a pair of bearings:

$$q = \left(\frac{5Q_v}{8} + \frac{T}{2c}\right)\frac{1}{D_e} + \frac{Q_h}{j - \sum w_h};$$

up to the level of the tops of any longitudinal bottom flange stiffeners above the bearing, or to a height j in the absence of any cut-outs and:

$$q = \left(\frac{5Q_v}{8} + \frac{T}{2c}\right)\frac{1}{D_e} + \frac{Q_h}{B_e};$$

d) In sections between pairs of bearing stiffeners above a single bearing:

$$q = \left(\frac{Q_v}{4}\right)\frac{1}{D_e} + \frac{Q_h}{j - \sum w_h};$$

up to the tops of any longitudinal stiffener, bottom flange stiffener cut-outs above the bearing or to a height j in the absence of any cut-outs;

or, above that level:

$$q = \left(\frac{Q_v}{4} + \frac{T}{s_s} - Q_T\right)\frac{1}{D_e} + \frac{Q_h}{B_e};$$

where:

$Q_v$, $Q_T$, $Q_C$ and $P_i$ are as defined in **Clause 11.6.2.3**;

$Q_{fv}$ is as defined in **Clause 11.5.2.3**;

$Q_h$, $Q_{bv}$, c are as defined in **Clause 11.5.2.4**;

$D_e$ and $B_e$ are the net depth and width of the diaphragm at the point under consideration;

j and $\Sigma w_h$ are as defined in **Clause 11.5.2.2**;

$s_s$ is the distance between stiffener centroids.

**Clause 11.6.2.4.2** The shear stress $\tau_p$ in the sections referred to in **Clause 11.6.2.4.1** (a), (b), (c) or (d) should be taken as:

$\tau_p = q/t_p$;

where:

$t_p$ is the thickness of the diaphragm plate in the panel under consideration.

**Clause 11.6.2.4.3** In sections other than those referred to in **Clause 11.6.2.4.1** (a), (b), (c), or (d), $\tau_p$ may be neglected.

**11.6.3** *Stresses in Diaphragm Stiffeners*

**11.6.3.1** *General*

Stresses in stiffeners should be determined in accordance with **Clause 11.6.3.2** to **11.6.3.4**, using the appropriate effective stiffener sections obtained from **Clause 11.4.4**.

The stiffener types: bearing, stub, primary intermediate and secondary intermediate, are as defined in **Clause 11.6.1**.

**11.6.3.2** *Vertical Stresses in Bearing Stiffeners*

a) The vertical stress σ1s in a bearing stiffener should be taken as:

$\sigma_{1s} = P_s / A_{se}$;

where:

$P_s$ is the total vertical force in the group of bearing stiffeners;

$A_{se}$ is the effective cross-sectional area of the group of bearing stiffeners, derived in accordance with **Clause 11.4.4**.

Both values are taken at the level under consideration.

b) In the absence of openings in the diaphragm between the group of bearing stiffeners and the adjacent web, the vertical force $P_s$ may be assumed to vary linearly from the value of the reaction at the bearing to the value of any reaction transmitted from the deck to the top of the bearing stiffener.

c) If there are any openings in the diaphragm between the group of bearing stiffeners and the adjacent web, no variation of load over the depth of such openings should be assumed. The variation over the remaining parts of the diaphragm should be assumed to be linear of constant slope. In the case of a diaphragm above a single bearing, an additional vertical stress $\sigma_{1sT}$ in a bearing stiffener should be taken as:

$\sigma_{1sT} = T_s y / I_{zse}$;

where:

$T_S$ is the value of the moment in the plane of the diaphragm on the group of bearing stiffeners;

$y$ is the horizontal distance of the stiffener under consideration from the centroidal axis, normal to the plane of the diaphragm, of the stiffener group (Refer **Fig. 11.1);**

$I_{zse}$ is the effective second moment of area of the stiffener group about the same centroidal axis, derived in accordance with **Clause 11.4.4**.

All values are taken at the point under consideration. $T_s$ may be assumed to vary linearly, from the torsional reaction above the bearing, to zero at the top flange level.

d) In the case of a diaphragm where stub stiffeners are used, the stress calculated as above may be reduced locally by including the area of such stiffeners, provided their connections to the diaphragm plate are adequate to transfer their share of the bearing reaction.

**11.6.3.3** *Bending Stresses in Bearing Stiffeners*

**11.6.3.3.1** The bending stress $\sigma_{bs}$ in a bearing stiffener due to an out-of-plane moment should be taken as:

$\sigma_{bs} = M_s z / I_{yse}$;

where:

$M_s$ is the proportion of the out-of-plane moment carried by the group of bearing stiffeners;

$z$ is the distance of the extreme fibre of the stiffener under consideration from the centroidal axis, parallel to the plane of the diaphragm, of the stiffener group **(**Refer **Fig. 11.1)**;

$I_{yse}$ is the effective second moment of area of the stiffener group about the same centroidal axis, derived in accordance with **Clause 11.4.4**.

All values are taken at the point under consideration.

**Clause 11.6.3.3.2** A proportion of the out-of-plane moment may be assumed to be carried by the flange longitudinal stiffeners, provided due account is taken of this in their design.

**Clause 11.6.3.3.3** Stub stiffeners should not be considered to carry any part of the out-of-plane moment carried by a bearing stiffener group unless they have an adequate out-of-plane shear connection to the bearing stiffeners and/or the box walls.

**11.6.3.4** *Equivalent Stress for Buckling Check*

**11.6.3.4.1** *General*

The equivalent axial stress $\sigma_{se}$ to be used in the buckling check of all stiffeners, should be taken as the maximum value within the middle-third of the length $l_s$ of the stiffener, calculated from:

$$\sigma_{se} = \sigma_a + \frac{1}{A_{se}}\left[\frac{\sigma_q t_s^2 t_p k_s}{a_{max}}\left(1 + \frac{\sum A_s}{l_s t_p}\right) + \tau_h t_p h_h\right]$$

where, for all stiffeners:

$A_{se}$ is the effective cross-sectional area of the stiffener derived in accordance with **Clause 11.4.4**;

$l_s$ is the length of the stiffener between points of effective restraint;

$t_p$ is the thickness of the diaphragm plate;

$k_s$ is obtained from **Fig. 11.6** using the slenderness parameter:

$\lambda = l_s / (r_{se}\varepsilon_s)$;

$r_{se}$ is the radius of gyration of the effective section of the stiffener about its centroidal axis parallel to the plane of the diaphragm, derived in accordance with **Clause 11.4.4**;

$\varepsilon_s = \sqrt{(250/f_{ys})}$;

$f_{ys}$ is the nominal yield strength of the stiffener;

$\Sigma A$ is the sum of the areas of all stiffeners which intersect the stiffeners being designed, within the length $l_s$ not including any adjacent diaphragm plate;

$\sigma_{p2}$ is derived in **Clause 11.6.2.3**, for the level being considered, and taken as positive when compressive;

$\sigma_{2s}$ is the average value of $\sigma_{p2}$ within the middle-third of the length $l_s$;

$\sigma_a$,$\sigma_q$, $\sigma_{max}$, $\tau_h$ and $h_h$ are defined as follows for the appropriate type of stiffener.

**11.6.3.4.2** *For Bearing Stiffeners*

$\sigma_a = \sigma_{1s} + \sigma_{1sT}$;

$\sigma_{1s}$ and $\sigma_{1sT}$ are as derived in **Clause 11.6.3.2**;

$\sigma_q = \sigma_{2s}$;

$a_{max}$ is the maximum spacing of vertical stiffeners which would ensure the adequacy of the diaphragm plate and any horizontal stiffeners, and may conservatively be taken as the actual spacing of vertical stiffeners;

$\tau_h$ and $h_h$ are taken as zero.

**11.6.3.4.3** *For all Intermediate Stiffeners*

$a_{max}$ is one-half of the sum of the panel widths on each side of the stiffener. Where the widths vary over the length $l_s$ the average value of the middle-third should be used;

$\tau$ is the average shear stress in the panels on either side of the stiffener;

$\tau_h$ is zero except in the case of the stiffeners framing openings, where $\tau_h$ is the shear stress which would occur in the plating adjacent to the stiffener if the opening had been fully plated;

$h_h$ is zero except in the case of the stiffeners framing openings where $h_h$ is the dimension of the opening parallel to the stiffener.

In calculating $a_{max}$ and $\sigma_q$ no account should be taken of any opening in the diaphragm adjacent to the stiffener (i.e. it should be assumed that a plate of thickness $t_p$ fills the opening).

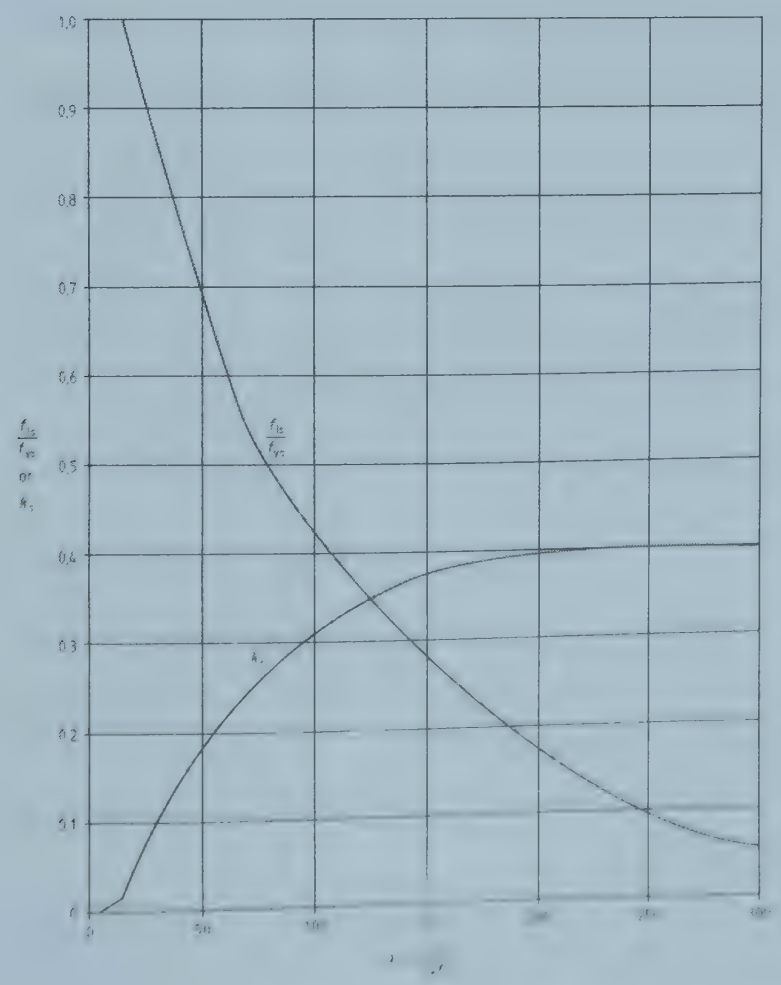



**Fig. 11.6 Parameters for the Design of Stiffeners [6]**

**11.6.3.4.4** *For Horizontal Intermediate Stiffeners only*

$\sigma_a = \sigma_{p2}$;

$\sigma_q = \tau$.

**11.6.3.4.5** *For Vertical Intermediate Stiffeners only*

$\sigma_a = 0$;

$\sigma_q = \tau + \sigma_{2s} + (\sigma_{2bmax} - \sigma_{2bmin})/12$;

$\sigma_{2bmax}$ and $\sigma_{2bmin}$ are the maximum and minimum values of $\sigma_{2b}$ derived as in **Clause 11.6.2.3**, within the length $l_s$ and taken as positive when compressive.

**11.6.4** *Yielding of Diaphragm Plate*

Plate panels between stiffeners, or between stiffeners and the box walls, should be designed such that at all points in every panel:

$$\sigma_{p1}^2 + \sigma_{p2}^2 - \sigma_{p1}\sigma_{p2} + 3\tau_{p2}^2 \leq \left(\frac{f_{yp}}{\gamma_{M0}}\right)^2;$$

where:

$\sigma_{p1} = \sigma_{1s} + \sigma_{1sT}$ for parts of plate panels forming part of the effective section of any bearing stiffener, or is the vertical in-plane stress due to local deck loads and change in flange slope, if relevant, for all remaining parts of plate panels;

$\sigma_{1s}$ is defined in **Clause 11.6.3.2**;

$\sigma_{1sT}$ is as derived in **Clause 11.6.3.2**, but with the value of y in that Clause taken as the dimension from the centroidal axis to the extreme fibre of the effective section of the stiffener group;

$\sigma_{p2}$ is defined in **Clause 11.6.2.3**;

$\tau_p$ is defined in **Clause 11.6.2.4**;

$f_{yp}$ is the nominal yield strength of the diaphragm plate.

**11.6.5** *Buckling of Diaphragm Plate*

**11.6.5.1** *Plate panels need not be checked for buckling provided that*

a) the cross-section of the girder is nominally rectangular;
b) the ratio of the depth of the diaphragm D to the minimum plate thickness $t_p$ is less than $100\varepsilon_p$, where $\varepsilon_p$ is as in **Clause 11.4.2.4;**
c) the overhang L (Refer **Fig. 11.4** or **Fig. 11.5**) from the outer edge of the bearing to the box web is less than D/2;
d) stiffening is limited to the bearing stiffeners themselves, and any member providing continuity of cross beam or cantilever flanges through the diaphragm;
e) there is no change in flange slope at the diaphragm.

**Clause 11.6.5.2** If any of the provisions of **Clause 11.6.5.1** are not satisfied, all plate panels should be verified taking into account buckling under the combined effect of stresses $\sigma_{p1}$, $\sigma_{p2}$ and $\tau_p$ as defined in **Clause 11.6.4**.

**Clause 11.6.5.3** A method for verifying plate panels is given in **Clause 11.9**. This may be used provided the following qualifications are met:

a) For panels adjacent to an inclined web the panel dimension should be taken as the maximum horizontal dimension of the panel.
b) A plate panel of non-constant thickness should be assumed to be of its minimum thickness throughout.
c) All plate panels adjacent to the box webs or flanges or to boundary stiffeners not more than $25t_p$ from the box walls or to large cut-outs should be treated as unrestrained. Other panels may be treated as restrained.
d) For a plate panel without horizontal stiffeners, bounded on three sides by the main beam web and the two main beam flanges, the shear stress coefficient $K_q$ should not be taken higher than:

$$\frac{1,3\ x\ 10^6}{f_{yp}}\left(\frac{t_p}{b}\right)^2\left[1+\left(\frac{b}{a}\right)^2\right];$$

where:

a and b are the length and width of the panel respectively;

$t_p$ and $f_{yp}$ are the thickness and nominal yield stress, respectively, of the diaphragm plate;

$\sigma_{p2}$ should be taken as the main longitudinal stress in the plate panel.

Hence, for the purposes of meeting the buckling criterion of **Clause 11.9**,

$\sigma_{p1}$ and $\sigma_{p2}$ as derived in **Clause 11.6.4** should be taken as $\sigma_2$ and $\sigma_1$.

**11.6.6** *Yielding of Diaphragm Stiffeners*

**11.6.6.1** A Bearing Stiffener Section should be Designed such that, at any Point along its Length:

$\sigma_{1s} + \sigma_{1sT} + \sigma_{bs} \leq f_{ys} / \gamma_{M0}$;

where:

$\sigma_{1s}$ and $\sigma_{1sT}$ are as defined in **Clause 11.6.3.2**;

$\sigma_{bs}$ is as defined in **Clause 11.6.3.3**;

$f_{ys}$ is the nominal yield strength of the stiffener.

**Clause 11.6.6.2** The bearing stress at the point of contact with a flange should be verified in accordance with **Clause 507, 509 & 510** of IRC:24. The out-of-plane buckling resistance of the transverse stiffener under transverse loads and shear force should be determined from **Clause 507, 509 & 510 of** IRC:24, using buckling curve c. When both ends are assumed to be

fixed laterally a buckling length of not less than $0,75h_w$ should be used. A larger value of $l$ should be used for conditions that provide less end restraint. If the stiffeners have cut outs at the loaded end, the cross-sectional resistance should be checked at this end.

**11.6.7** *Buckling of Diaphragm Stiffeners*

The stiffener section should be such that, at any point within the middle-third of the length of the stiffener:

$$\frac{\sigma_{se}}{f_{ls}} + \frac{\sigma_{bs}}{f_{ys}} \leq \frac{1}{\gamma_{M1}};$$

where:

$\sigma_{se}$ is as defined in **Clause 11.6.3.4**;

$\sigma_{bs}$ is as defined in **Clause 11.6.3.3** for a bearing stiffener, or is taken as zero for an intermediate stiffener;

$f_{ls}$ is obtained from **Fig. 11.6** using the slenderness parameter

$\lambda = l_s / (r_{se}\varepsilon_s)$;

$l_s$ is the length of the stiffener between points of effective restraint;

$r_{se}$ is the radius of gyration of the effective section of the stiffener about its centroidal axis parallel to the plane of the diaphragm, derived in accordance with **Clause 11.4.4;**

$f_{ys}$ is the nominal yield strength of the junction section.

## 11.7 Diaphragm/Web Junctions

**11.7.1** *General*

The diaphragm/web junction should be designed as a stiffener to the box web, spanning between box flanges, unsupported in the plane of the diaphragm, and with effective section derived as in **Clause 11.4.5**.

**11.7.2** *Loading Effects to be Considered*

The junction should withstand the effects of the following.

a) All loads transmitted to the diaphragm from the cross beams and/or cantilevers in the plane of the diaphragm. Such loads should be assumed to be applied at the centroidal axis of the effective section, and to vary linearly from a maximum at the top of the junction, to zero at the bottom.
b) Any forces resulting from tension field action in the adjacent web panels. Such forces should be assumed to be applied in the plane of the box web, and to be constant over the height of the junction.
c) An equivalent axial force representing the destabilizing influence of the web. This force should be assumed to be applied at the centroidal axis of the effective section, and to be constant over the height of the junction.

**11.7.3** *Strength of Diaphragm/Web Junction*

*11.7.3.1* The maximum stress at any point on the cross-section of the junction, at any section in its length, should not exceed:

$f_{ys} / \gamma_{M0}$

where:

$f_{ys}$ is the nominal yield strength of the junction section.

*11.7.3.2* The effective junction section should be such that:

$$\frac{P}{A_{se} f_{ls}} + \frac{M}{W_{se} f_{ys}} \leq \frac{1}{\gamma_{M1}};$$

where:

P and Mare, respectively, the maximum force on the effective junction section and the maximum moment about the centroidal axis parallel to the web due to all the effects specified in **Clause 11.7.2**, within the middle-third of the length of the junction;

$A_{se}$ is the effective area of the junction section (Refer **Clause 11.4.5**);

$W_{se}$ is the lowest section modulus of the effective junction section about the centroidal axis parallel to the web (Refer **Clause 11.4.5**)

$f_{ls}$ is obtained from **Fig. 11.6** using the slenderness parameter:

$\lambda = l_s / (r_{se} \varepsilon_s)$;

$l_S$ is the total length of the junction section;

$r_{se}$ is the radius of gyration of the effective section(stiffener and plate) about the centroid axis parallel to the web, derived in accordance with **Clause 11.4.5**;

$f_{ys}$ is the nominal yield strength of the junction section

**11.7.4** *Junction Restraint Provided by Diaphragm Stiffeners*

Diaphragm/Web Junctions should be designed in accordance with **Clause 11.7.1** to **11.7.3**, except that full width horizontal stiffeners in the diaphragm may be assumed to offer restraint to the junction in the plane of the diaphragm, provided that the equivalent axial stress $\sigma_{se}$ in such stiffeners (Refer **Clause 11.6.3.4**) is increased by an amount equal to:

$0{,}025\ P / (n\ A_{se})$

where:

*P* is as defined in **Clause 11.7.3**;
*n* is the number of full width horizontal stiffeners;
$A_{se}$ is the effective area of the horizontal stiffeners, derived in accordance with **Clause 11.4.4**.

In this case $l_S$ in **Clause 11.7.3** may be taken as the distance between such stiffeners.

## 11.8 Continuity of Cross Beams and Cantilevers

### 11.8.1 *General*

When continuity of crossbeams and cantilevers is provided in the plane of a diaphragm, in accordance with **Clause 11.2.3**, that portion within the box walls should be in accordance with the following.

a) The force in the member providing continuity to the bottom flange of the transverse member should be taken as the moment in the transverse member at the box wall divided by the distance between the mid-plane of the top and bottom flanges of the member. If the force is different at the two box walls a linear variation along the length may be assumed.

b) If the member providing the continuity in a) is also required as a horizontal stiffener for a diaphragm designed in accordance with **Clause 11.6**, it should be designed to withstand, in addition to the load given in a), an axial force equal to $A_{se}\sigma_{se}$.
where:
$A_{se}$ is the effective cross-sectional area of the continuity member derived in accordance with **Clause 11.4.4**;
$\sigma_{se}$ is as specified in **Clause 11.6.3.4**.

c) The member providing the continuity in a) should be designed as a compression member in accordance with **Clause 507, 509 & 510** of IRC:24, and should be assumed to be unrestrained out of the plan of the diaphragm unless provided with effective intermediate restraint. If these restraints are provided by bearing or primary vertical diaphragm stiffeners, such stiffeners should each be designed to resist, in addition to all other forces given in **Clause 11.6.3**, a force equal to 2.5% of the maximum axial load in the continuity member including that given in b), if appropriate. This force should be applied, out of the plane of the diaphragm, at the point of intersection of the continuity member and the stiffener providing the restraint. The stiffener should be designed to satisfy the criterion:

$$\frac{\sigma_{se}}{f_{ls}} + \frac{\sigma_{bs} + \sigma_{b2}}{f_{ys}} \leq \frac{1}{\gamma_m}$$

where:

$\sigma_{b2}$ is the bending stress induced in the stiffener by the above force, taken as the maximum value within the middle-third of the lengths of the stiffener;

$\sigma_{bs}$, $f_{ls}$, $\sigma_{se}$ and $f_{ys}$ are as defined in **Clause 11.6.7**.

## 11.9 Buckling of Panels in Diaphragms

### 11.9.1 *General*

This section may be used for verification of panels taking into account buckling. It may only

be used subject to the limitations in **Clause 11.2, Clause 11.6.5.2**, and **11.6.5.3**.
In particular, note that for use with this method:

$\sigma_{p2} = \sigma_1$ (relates to $K_1$, $m_1$, etc.);
$\sigma_{p1} = \sigma_2$ (relates to $K_2$, $m_2$, etc.).

**11.9.2** *Restraint of Panels*

Effective panel restraint should be defined in **Clause 11.5.3** for calculation of the buckling coefficients $K_1$, $K_q$, $K_b$ and $K_2$ required in **Clause 11.9.3**.

**11.9.3** *Buckling Coefficients*

*11.9.3.1* The coefficients $K_1$, $K_q$, $K_b$ and $K_2$ should be obtained from **Clause 11.9.3.2, 11.9.3.3, 11.9.3.4 & 11.9.3.5** respectively, with the panel assumed to be restrained or unrestrained in-plane as determined from **Clause 11.9.2**.

*11.9.3.2* For axial coefficients, when the stress is compressive, $K_1$ should be taken as the greater of the values obtained as follows:

Either:

a) from **Fig. 11.8 (a)** using curve 1 or 2 as appropriate, with:

$$\lambda = 0.81 \frac{b}{t_p} \sqrt{\frac{f_{yp}}{250}}$$

unless λ is less than 24, when:

$$K_1 = \left(\frac{t_p}{b}\right)^2 \frac{204\,500}{f_{yp}};$$

or:

b) from **Fig. 11.8 (a)** curve 3 with:

$$\lambda = 0.81 \frac{a}{t_p} \sqrt{\frac{f_{yp}}{250}}$$

unless λ is less than 4,33, when:

$$K_1 = \left(\frac{t_p}{a}\right)^2 \frac{6\,600}{f_{yp}};$$

where:

$t_p$ is the diaphragm thickness;

$f_{yp}$ is the nominal yield stress of the material;

*a* and *b* are as defined in **Fig. 11.8(a)**.

When the stress is tensile, $K_1$ should be taken as 1.0

*11.9.3.3* The shear coefficient $K_q$ should be taken from **Fig. 11.8(b),**

unless:

$$\frac{b}{t_p} < 35\sqrt{\left(1+\left(\frac{b}{a}\right)^2\right)};$$

when:

$$K_q = \left(\frac{t_p}{b}\right)^2 \frac{435\,000\left[1+\left(\frac{b}{a}\right)^2\right]}{f_{yp}};$$

where:

$t_p$ and $f_{yp}$ are as defined in **Clause 11.9.3.2;**

*a* and *b* are defined in **Fig. 11.8 (b).**

*11.9.3.4* The bending coefficient $K_b$ should be taken from **Fig. 11.8 (c)**.

*11.9.3.5* Transverse coefficient $K_2$ should be taken as the greater of the values obtained as follows:

either:

a) from **Fig. 10.8 (a)** using curve 1 or 2 as appropriate, with:

$$\lambda = 0{,}81\frac{a}{t_p}\sqrt{\frac{f_{yp}}{250}};$$

unless $\lambda$ is less than 24, when;

$$K_2 = \left(\frac{t_w}{a}\right)^2 \frac{204\,500}{f_{yp}}$$

or:

from **Fig. 11.8 (a)**, curve 3 with:

$$\lambda = 0.81\frac{b}{t_p}\sqrt{\frac{f_{yp}}{250}}$$

unless $\lambda$ is less than 4.33, when:

$$K_2 = \left(\frac{t_p}{a}\right)^2 \frac{6\,600}{f_{yp}};$$

where:

$t_p$ is the diaphragm thickness;

$f_{yp}$ is the nominal yield stress of the material;

*a* and *b* are as defined in **Fig. 11.8 (a).**

When the stress is tensile, $K_2$ should be taken as 1.0

**11.9.4** *Interaction Buckling Criterion*

The following condition should be satisfied for each panel:

$m_c + m_b + 3m_q \leq 1$

$m_c = \sqrt{m_1 + m_2}$ for positive values of $(m_1 + m_2)$;

$m_c = -\sqrt{-m_1 + m_2}$ for negative values of $(m_1 + m_2)$;

$m_1 = \left(\frac{\sigma_1 \gamma_m}{f_{yp} K_1}\right)^2$ to be taken as negative if $\sigma_1$ is tensile

$m_2 = \left(\frac{\sigma_2 \gamma_m}{f_{yp} K_2}\right)^2$ to be taken as negative if $\sigma_2$ is tensile

$m_b = \left(\frac{\sigma_b \gamma_m}{f_{yp} K_b}\right)^2$

$m_q = \left(\frac{\tau_p \gamma_m}{f_{yp} K_q}\right)^2$

where:

$\sigma_1$ is the mean longitudinal stress on a cross-section of the panel, considered positive if compressive. The algebraically higher of the two values on the opposite edges of a panel should be used;

$\sigma_2$ is the transverse stress, considered positive if compressive. The algebraically higher of the two values on the opposite edges should be used;

$\sigma_b$ is the maximum longitudinal stress due to in-plane bending of the individual panel (Refer **Fig. 11.7**);

$\tau_p$ is as defined in **Clause 11.6.2.4**;

$K_1$, $K_q$, $K_b$ and $K_2$ are coefficients derived in accordance with **Clause 11.9.3.2** to **11.9.3.4**;

$f_{yp}$ is the nominal yield stress of the diaphragm material.

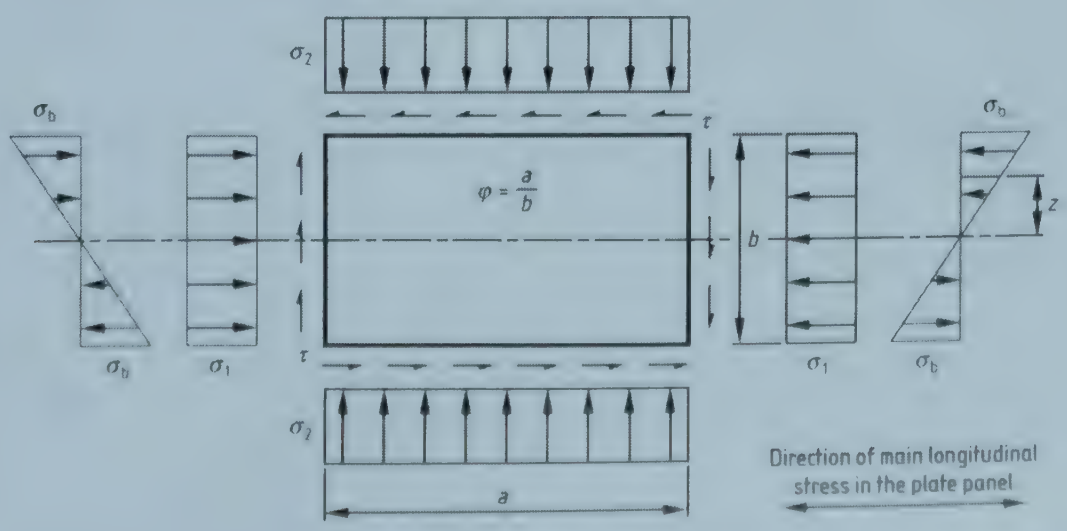
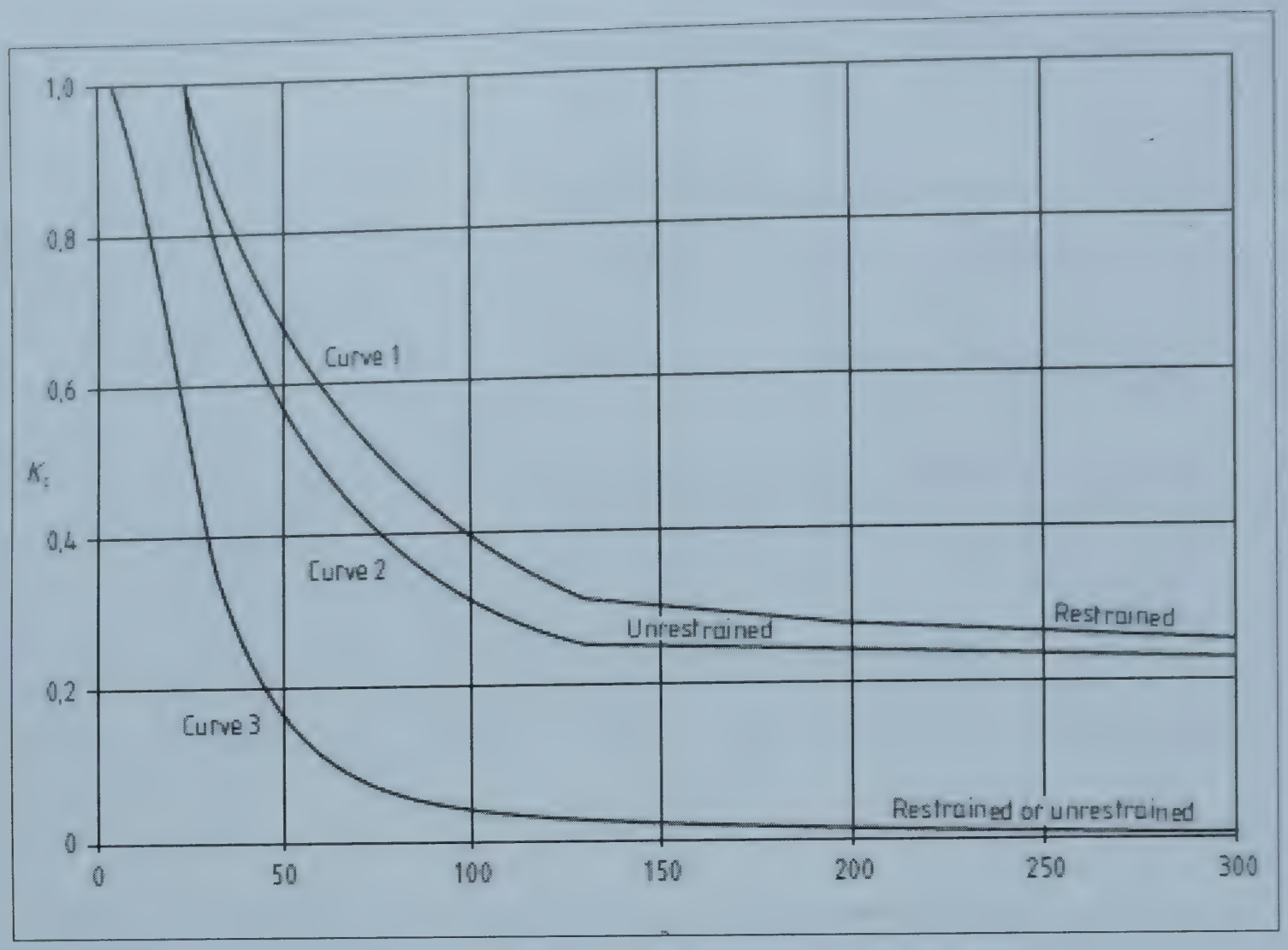
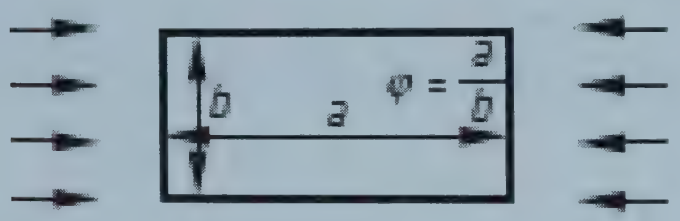



**Fig. 11.7 Stresses on Panels**




**NOTE 1**

**For stresses shown as follows**




$K_2$ is the greater value of $K_e$ obtained from curve 1 or 2, as appropriate with:

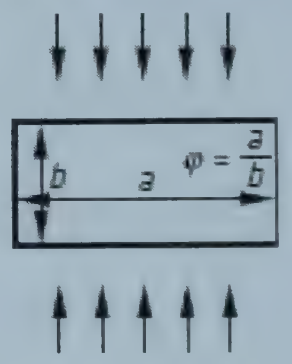
$$\lambda = 0{,}81\frac{b}{t_p}\sqrt{\frac{f_{yp}}{250}};$$

or curve 3 with:

$$\lambda = 0{,}81\frac{a}{t_p}\sqrt{\frac{f_{yp}}{250}};$$

**NOTE 2**

**For stresses shown as follows**



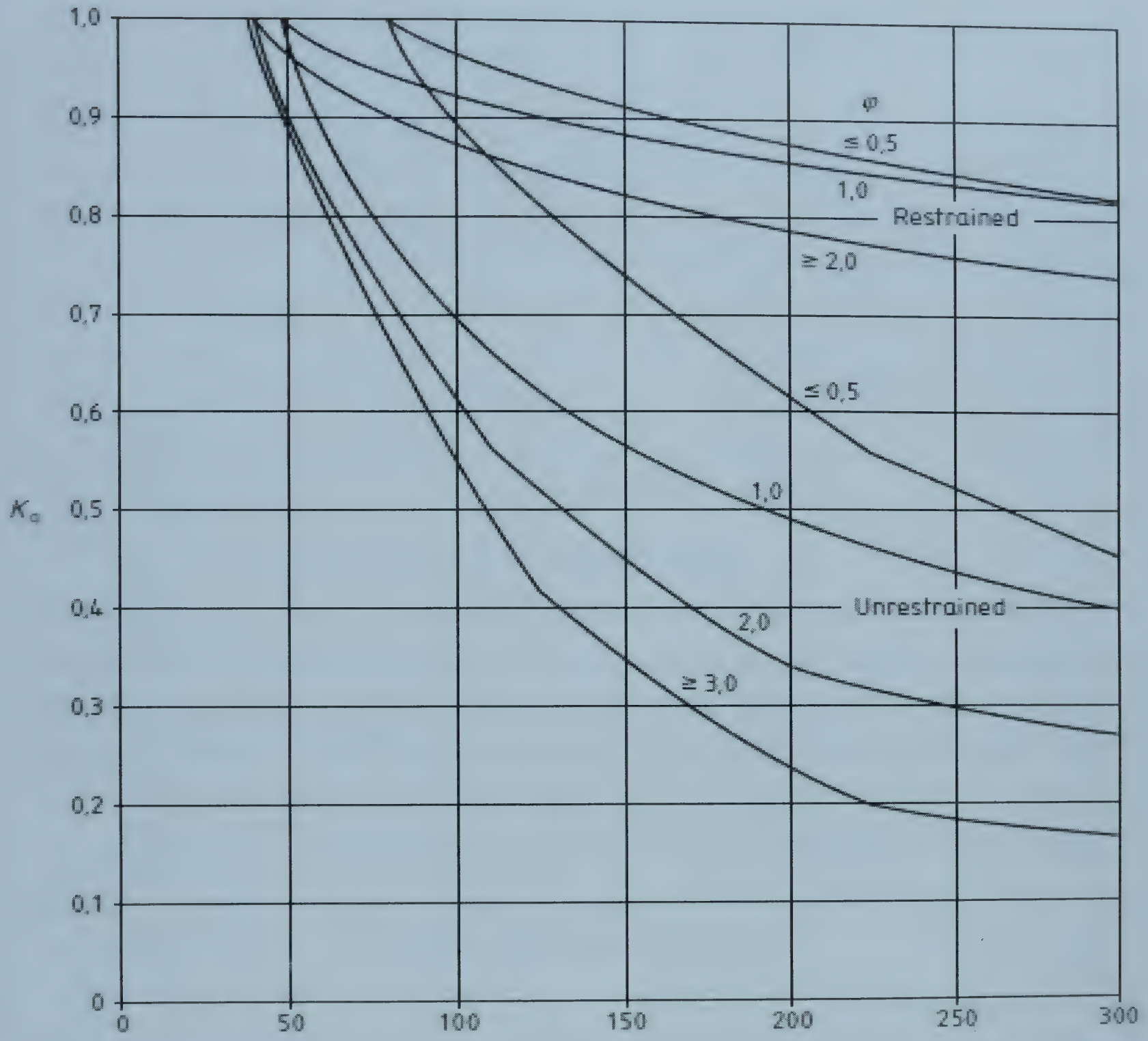

$K_2$ is the greater value of $K_e$ obtained from curve 1 or 2, as appropriate with:

$$\lambda = 0{,}81\frac{a}{t_p}\sqrt{\frac{f_{yp}}{250}}$$

or curve 3 with:

$$\lambda = 0{,}81\frac{b}{t_p}\sqrt{\frac{f_{yp}}{250}}$$

**Fig. 11.8 (a) Buckling Coefficients $K_1$, $K_2$, $K_q$ and $K_b$ [6]**




$$\lambda = 0.81 \frac{b}{t_p} \sqrt{\frac{f_{yp}}{250}}$$

*NOTE 1 For intermediate values of φ $K_q$ may be obtained by linear interpolation between adjacent values of φ for this purpose a curve may be extended beyond $K_q = 1$.*

*NOTE 2 φ = a/b shown as follows:*

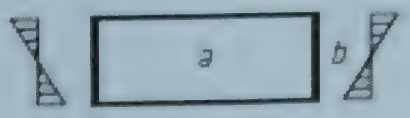
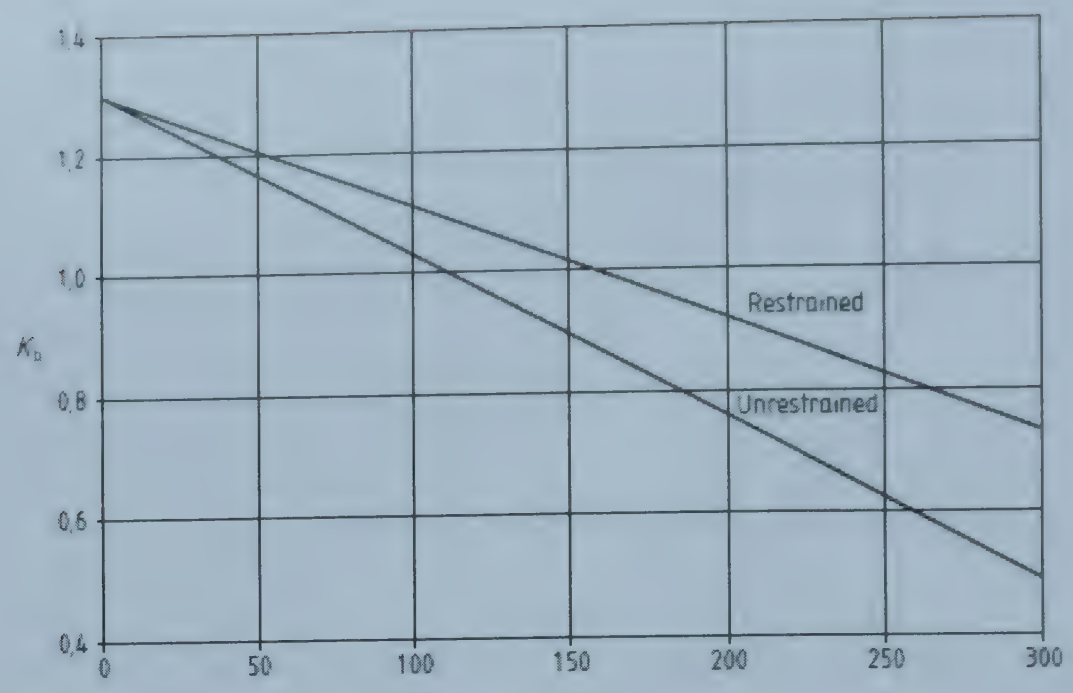



**Fig. 11.8(b) Values of $K_q$ [6]**




$$\lambda = 0.81 \frac{b}{t_p} \sqrt{\frac{f_{yp}}{250}}$$

**Fig. 11.8(c) Values of $K_b$ [6]**

### 11.10 Plate Girder Type End Cross Girder / Diaphragm

For multi-girder spans, generally the end cross girder is placed on the bearings. It also acts as end diaphragm of the individual box girders. It can be designed as a composite plate girder following the provisions of IRC: 22/ 24. Besides the bearing stiffeners at Bearing and Jacking location, the web plate shall be adequately stiffened following the provisions of IRC:24. The openings in the web plate shall be proportioned as per the provisions of **Clause 11.2.7**.

## 12. CROSS FRAMES

### 12.1 General

This section provides guidance on the design of ring or braced intermediate cross frames provided in box girders to restrict distortion of the cross-section or to control the effects due to distortional warping, subject to the limitations and provisions of **Clause 12.2**. The design of plated intermediate diaphragms is not covered, but they may be used provided special analysis is undertaken.

### 12.2 Limitations

**12.2.1** *Girder Layout*

Girders should be nominally symmetric (i.e. ignoring cross fall or super elevation), of rectangular or trapezoidal cross-section, and with webs in single planes inclined at less than 20° from the vertical. Girders should be of single cell form, with or without interconnecting cross members and cantilevers.

**12.2.2** *Cross Frames*

The cross frames should be in a single plane, within ±5° of the normal to the axis of the girder in elevation, within ±10° in plan, and within ±5° of a vertical plane.

**12.2.3** *Corner Stiffening*

Each corner of the cross frame (i.e. at the intersections of flange and we b of main box) should be stiffened as necessary to withstand the bending moments acting at the corner. In the absence of such stiffening, no effective width of box web or flange (Refer, **Clause 12.4.1**) should be assumed to act with the cross frame at this point.

**12.2.4** *Stiffeners*

Where distortion due to torque applied to a girder between cross frames is to be resisted by the box walls, the cross frames should have dimensionless stiffness of at least that defined in **Clause 9.3.4.3**.

## 12.3 Ring Cross Frames

**12.3.1** *Effective Section of Transverse Members*

The effective section of transverse members may be taken as the gross area comprising the stiffener plus a width of plate equal to 15εt but not more than the actual dimension available, on each side of the stiffener avoiding any overlap of contributing parts to adjacent stiffeners.

**12.3.2** *Strength of Ring Cross Frame Members*

Components of ring cross frames attached to the web or flange plates of main beams should satisfy the strength provisions of transverse members/stiffeners as appropriate.

## 12.4 Braced Cross Frames

**12.4.1** *General*

Braced cross frames should be designed to satisfy the provisions of **Clause 12.3, Clause 12.4.2** and **12.4.3**.

**12.4.2** *Design Forces and Moments to be Considered*

Additional account should be taken of bending moments induced by eccentricities at connections, incomplete triangulation and application of loading other than at points of intersection between members.

**12.4.3** *Design of Braced Cross Frame Members*

The design of compression and tension members of braced cross frames, not attached to the web or flange plate of main beams, should be in accordance with **Clause 506** & **507** of IRC:24 respectively.

# 13. DESIGN OF STIFFENERS

Refer **Clause 509.7** of IRC:24.

Additionally, the minimum requirements for longitudinal stiffeners shall be as follows:

The requirements concerning torsional buckling are as follows:

Unless a more advanced method of analysis is carried out in order to prevent torsional buckling of stiffeners with open cross-sections, the following criterion should be satisfied:

$$\frac{I_T}{I_P} \geq 5.3 \frac{f_y}{E}$$

where,

$I_p$ is the polar second moment of area of the stiffener alone around the edge fixed to the plate;

$I_T$ is the St. Venant Torsional constant for the stiffener alone.

Where warping stiffness is considered stiffeners should either fulfil the above criterion or the criterion:

$\sigma_{cr} > \theta f_y$

where,

$\sigma_{cr}$ is the elastic critical stress for torsional buckling not considering rotational restraint from the plate;

$\theta$ is a parameter to ensure class 3 behaviour. The value $\theta = 6$ is recommended.

Discontinuous longitudinal stiffeners that do not pass through openings made in the transverse stiffeners or are not connected to either side of the transverse stiffeners should be:

- used only for webs (i.e. not allowed in flanges);
- neglected in global analysis;
- neglected in the calculation of stresses;
- considered in the calculation of the effective widths of web sub-panels;
- considered in the calculation of the elastic critical stresses.

Strength assessments for stiffeners should be performed.

If longitudinal stiffeners are taken into account in the stress analysis, they should be checked for direct stresses for the cross-sectional resistance.

# 14. REFERENCES

The following Codes and other publications may be referred to for further information on this topic.


1. IRC:24 "Standard Specifications and Code of Practice for Road Bridges, Steel Road Bridges (Limit State Method) (Third Revision)"
2. IRC:22 "Standard Specifications and Code of Practice for Road Bridges, Section VI – Composite Construction (Limit States Design) (Third Revision)"
3. BS 5400-Part 3-2000 "Steel, Concrete and Composite Bridges - Code of Practice for Design of Steel Bridges"
4. BS EN 1993-1-5:2006 "Design of Steel Structures (Part 1)"
5. BS EN 1993-2: 2006 "Design of Steel Structures (Part 2)"
6. PD 6695-2:2008 "Recommendations for the Design of Steel Bridges"
7. SCI Publication P140: Design Guide for Composite Box Girder Bridges
8. SCI Advisory — AD 331: Open top box girder for bridges
9. Steel Bridges: Conceptual and Structural Design of Steel and Steel Concrete Composite Bridges – M. Hirt & J. Lebet
10. Bridge Engineering Handbook – Construction and Maintenance – WF Chen & L Duan
11. Tapered Box Girders of Deformable Cross Section - Vladimir Kristek - Journal of the Structural Division, ASCE, Vol. 96, No. ST8, Aug.1970, pp. 1761 - 1793.

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the Code/Guidelines/ Manual etc. from the date specified therein)

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